JOURNAL

of the

AMERICAN CONCRETE INSTITUTE

Published monthly except July and August by the
American Concrete Institute under the authority of the Board of Direction

EDITED BY THE SECRETARY

with the direction of the PUBLICATIONS COMMITTEE: W. A. SLATER, chairman; P. H. BATES, ARTHUR R. LORD, J. C. PEARSON, A. N. TALBOT, BENJAMIN WILK, HARVEY WHIPPLE, Secretary.

Papers: Original papers, furthering the objects of the society, are invited. They should be addressed to the Secretary, and their consideration by the Publications Committee will be facilitated if: (1) a brief but comprehensive outline or summary precedes or at least accompanies the completed paper; (2) manuscript and photographs are available in duplicate or triplicate; (3) original tracings or other drawings, from which copies are readily made, are available.

Effort will be made to advise authors promptly of decisions as to availability for publication. Acceptance of a paper does not imply the acquiescence of critics (whose advice is sought prior to acceptance) or of the Institute in opinions which it expresses nor the acceptance of data or practice which it records. Institute authority attaches only to standards adopted as provided in the By-Laws. The publication of a committee report implies merely the cooperative effort of an appointed group within the Institute. Acceptance of a paper or report for publication does not commit the Program committee in reference to the annual convention.

Subscription price $12.50 per year payable in advance. To members, $10.00 per year, included in the annual dues.

Bound volumes 1 to 22 of *Proceedings of the American Concrete Institute* (1905 to 1926)—$8.00 each so far as available; Volumes 23, 24, 25 (1927-1929) $10.00 each. To members $5.00 each. Volume 26 (1930) consisting of JOURNAL issues 1-8, November 1929-June 1930, bound without *News Letter* pages to non-members and non-subscribers, $12.50. Subscribers and members may obtain the volume (in addition to their monthly issues) at the end of the volume year at $6.00, so long as a limited supply of over-copies permits. Members may insure availability of not more than two copies of each volume at a special price when they place their orders in advance of the first JOURNAL issue of each volume.

Publication address: 2970 West Grand Boulevard, Detroit, Michigan. Entered at the Post Office at Detroit, Michigan, as mail of the second class under the provisions of the Act of March 3, 1879.

Copyrighted, 1930
American Concrete Institute.

AMERICAN CONCRETE INSTITUTE

Founded 1905; incorporated 1906 (National Association of Cement Users); charter amended 1913. (American Concrete Institute.)

BOARD OF DIRECTION

President

D. A. ABRAMS (Term expires Feb. 1931)

Vice-Presidents

S. C. HOLLISTER (Term expires Feb. 1931)
ARTHUR R. LORD (Term expires Feb. 1932)

Secretary and Treasurer

HARVEY WHIPPLE (Term expires Feb. 1931)

Directors:

First District—R. B. YOUNG (Term expires Feb. 1932)
Second District—CLOYD M. CHAPMAN (Term expires Feb. 1932)
Third District—J. C. PEARSON (Term expires Feb. 1931)
Fourth District—P. H. BATES (Term expires Feb. 1931)
Fifth District—F. R. McMILLAN (Term expires Feb. 1931)
Sixth District—RAYMOND E. DAVIS (Term expires Feb. 1932)

Past Presidents

RICHARD L. HUMPHREY Deceased (1905-14)
LEONARD C. WASON (1915-16)
WILLIAM K. HATT (1917-19)
*HENRY C. TURNER (1920-21)
*WILLIAM P. ANDERSON (1922-23)
*A. E. LINDAU (1924-25)
*M. M. UPSON (1926-27)
*E. D. BOYER (1928-29)

*The five latest, living past presidents are members of the Board.

VOL. 2—NO. 4 DECEMBER 1930 (*Proceedings Vol. 27*)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

News Letter

FEBRUARY 24-26

REGISTRATION headquarters for the 27th annual convention will be set up Monday morning, Feb. 23, at the Schroeder Hotel, Milwaukee. Many concrete products manufacturers will be arriving that day for joint meetings of the Concrete Masonry Association (formerly National Concrete Products Association) and the Wisconsin Concrete Products Association.

The Institute's first session will be at 2 o'clock Tuesday afternoon, Feb. 24. There will be sessions Tuesday evening, Wednesday afternoon, Thursday morning, Thursday afternoon. The dinner will be Wednesday evening.

WATERPROOFING

P. H. BATES will tell the convention about an investigation of waterproofing at the Bureau of Standards.

The discussion will cover some preliminary work done in developing a concrete which could be made in the laboratory and produce a specimen which would show permeability to water under a head. After having developed a concrete, which was necessary since even ordinary well-made concretes will give impermeable test specimens, practically all of the integral waterproofing compounds available on the market are incorporated into the specimens. After aging, the specimens in the form of discs were submitted to a head of 20 pounds continuously for one year; the amount of water coming through the specimens being measured at intervals. At the end of the year they were removed, the surface exposed to the water brushed off and the permeability determined over a period of a month. The specimens were then again removed, allowed to dry, and the permeability again determined over a period of one month. Three by six cylinders have also been made into which the compounds were incorporated, and the strength of these as well as their absorption will be determined at intervals in comparison with con-

crete similarly aged but not containing the compounds.

Tests of a somewhat similar nature have been made of waterproofings intended to be used as coatings. Tested under such conditions, the results show that many of the compounds will reduce the degree of permeability.

LONG-TIME TESTS OF CONCRETE

M. O. WITHEY, Professor of Mechanics, University of Wisconsin, will present results and conclusions of "Some Long-Time Tests of Concrete." This contribution will report two series of tests made at the University of Wisconsin primarily to ascertain the effects of age and curing conditions on the strength of concrete mixes of common proportions. The 1910 program covering compression tests on 450 6x12 in. concrete cylinders of 1:3:6 and 1:2:4 proportions subjected to three types of curing conditions, one of which is outdoor exposure, has been planned to run fifty years. The data obtained during the first twenty years is presented. In 1923 a program involving the fabrication of 1800 6x12 in. cylinders and a large number of auxiliary specimens which are to be tested over a one hundred year period was initiated. The results covering the tests made during the first five years are reported. The later series of tests cover four variables in brand of cement, three variables in coarse aggregate, four variables in proportions, and the effect of inside and outside curing conditions.

PROPERTIES OF MASS CONCRETE

THE WORK of Committee 108, Properties of Mass Concrete, touches the quick of problems arising in large undertakings—dams for instance—of which the far west has a considerable share. Raymond E. Davis, Professor of Civil Engineering, University of California, as chairman of the committee, called a meeting at the Faculty Club, Nov. 13. Eighteen committee members and guests were present. They considered and discussed: A summary by Prof. G. E. Troxell of what is known of the behavior of concrete in mass construction; the results of investigations by the Los Angeles County Flood Control District, presented by E. C. Eaton, chief engineer, and R. W. Carlson, testing engineer; studies of the effect of cement fineness as reported by Hubert Woods, chemist, Riverside Cement Co.; and experiences and observations by I. E. Houk, senior engineer, U. S. Bureau of Reclamation, Denver, D. C. Henny, consulting engineer, Portland Ore., and A. J. Wiley, consulting hydraulic engineer, Boise, Idaho. Members of the committee present: R. E. Davis, chairman; P. H. Bates, F. W. Hanna, D. C. Henny, W. A. Perkins, G. E. Troxell, A. J. Wiley; Guests: I. E. Houk, E. C. Eaton, S. B. Morris, chief engineer, Pasadena Water Department, Pasadena; W. L. Huber, consulting engineer, San Francisco; N. C. Bowers, vice-president McGraw Hill Publishing Co. and Pacific Coast Editor *Engineering News Record*; G. W. Hawley, California Deputy State Engineer in Charge of Dams, of Sacramento; R. W. Carlson, S. E. Hutton, Engineer, Pacific Coast Cement Co., Seattle.

There was further discussion of a comprehensive program of field and laboratory tests to clear up the many gaps in existing data. The committee will probably be enlarged and several sub-committees formed for the undertaking.

COMMITTEES

109—Summary of Plastic Flow—Raymond E. Davis, author-chairman. H. E. Davis and J. R. Shank have been appointed and have accepted critic membership.

206—Welded Wire Fabric as a Reinforcing Material—R. D. Bradbury, author chairman. The assignment is to discuss in summary, available information on: Properties of the wire used by various manufacturers; effect of welding on properties of material at and adjoining intersections; trade standards adopted; available stock and special-order sizes, weights, areas and protective finishes; necessary end and side laps for full continuity as reinforcing, considering all kinds of stresses involved; comparison of welded mesh and plain or deformed bars as to bond, size and spacing of cracks, shrinkage, etc., at various steel stresses; cost and cost of placing; special treatment at openings; welded wire stirrups; effect of same amount of same metal as between larger bars at larger spaces and smaller bars or wire at smaller spaces; supporting wire mesh in slabs at proper elevation; catenary versus ordinary continuous slab action; can catenary action take place without cracks due to failure of plain concrete where mesh drops away from upper surface of slab rapidly on either side of girder? use of mesh as wrapping material in fireproofing, ordinary aggregates and fire-spalling aggregates; and so on.

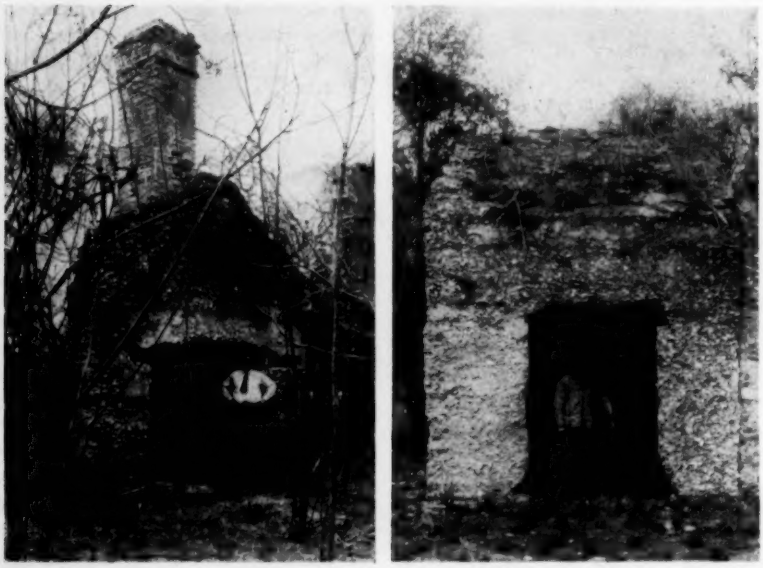
607—Use of Bulk Cement—Herbert Coffman, author-chairman. The following have accepted appointment as critic members: Peter F. Stauffer (The Fuller Co., Catasaugua, Pa.), Malcolm H. Ulman (Asst. Engr. of Tests, Pennsylvania Dept. of Highways), Andrew Seifert (U. S. Bureau of Public Roads), Fred C. Wilcox, I. E. Burks, A. G. Watt (Lehigh Portland Cement Co.)

709—Specifications for Concrete Burial Vaults—W. D. M. Allan, chairman. Committee being organized; the task implied in the title of the committee.

THEY PROPOSED 134 NEW MEMBERS

NINETY-EIGHT members of the Institute have developed results from active membership effort—67 of them have sponsored one new member each since Feb, 1, 1930 (to Dec. 1, 1930); eleven members have each sponsored two; four have nominated three each; three are credited with four each; one with six; one with seven and one with eight. That makes a total of 134. Those with two or more to their credit are listed as follows:

W. D. M. Allan	8
F. R. McMillan	7
Raymond E. Davis	6
D. A. Abrams	4
Miles N. Clair	4
P. G. Gauthier	4
Einar Christensen	3
J. C. Pearson	3
F. E. Richart	3
M. M. Upson	3
Charles S. Barr	2
Cloyd M. Chapman	2
Herbert Coffman	2
Herman Frauenfelder	2
H. J. Gilkey	2
Francisco Gomez-Perez	2
Master Builders Co.	2
J. H. Murdough	2
H. C. Turner	2
W. F. Way	2
R. B. Young	2



MISS GLEASON'S ANTIQUES ON DATAW ISLAND

At the left the chimney place built 1690 by Bonun Sams, an Oxford graduate from a Welsh family; at the right the smoke house built by Barnwell Sams, 1740—both built of the only kind of "concrete" the early Carolina province knew.

KATE GLEASON FINDS OLD "CONCRETE" AT SCENE OF HER LATEST WORK

FROM A LETTER BY KATE GLEASON

Miss Gleason, who in 1922, related to an Institute convention audience some of her experiences in building concrete houses in her East Rochester development† has since accumulated experiences in other scenes, none more interesting perhaps than those of the last two years at the Sea Islands of South Carolina.*

HOW KIND of you to ask for information and give me a chance to extol my latest. I am mailing you a book about Beaufort or rather about this part of the Sea Islands and you will notice the Spaniards and French tried to get in here as early as 1525, but the Indians wouldn't let the Spaniards stay and the Spanish wouldn't let the French stay. But the British were more successful and I am sending you some photos of a couple of antiques I bought with one of the islands. There is a "poured concrete"‡ chimney piece put up in 1690. You will notice that they were wise enough to make the part immediately affected by the heat of brick. And there is a smoke house of "poured

*Only woman holding A. C. I. membership.

†"Small Concrete Houses at Rochester, N. Y.", A. C. I. *Proceedings*. Vol. 18, p. 124.

‡Miss Gleason assumes poetic license. It was "concrete" in the broadest sense of aggregation. The book on Beaufort county, she so kindly sent, says "tabby" (shells; oyster-shell lime and sand).—EDITOR.

concrete," roof and all, dating from 1740. The aggregate used is oyster shells and beside the cement I notice they used a kind of red clay something like brick clay. They evidently knew the virtue of lots of tamping and as this plantation had about 1,000 slaves they could afford to keep at it. Almost 250 years for the chimney piece and almost 200 years for the smoke house! I don't know the dates of other old poured concrete buildings but mine can't be far from the record for the United States.

I've been working here for the last couple of years. I heard from a Beaufort girl who was helping me in California that this neighborhood was pitifully poor and with my vivid memory of how uncomfortable it is to be poor, I came down to see whether I could help. My first effort was to stucco a pre-revolutionary house and put a couple of bathrooms in it. Then I gave a dinner party and one enthusiastic speaker in extolling my work mentioned the fact that Christ had raised Lazarus from the Dead. But Miss Gleason was raising Beaufort from the Dead. Even the banquet speakers of the American Concrete Institute don't do much better for me than that.

I built a poured concrete tavern here last February. Every room with a bath which is a record for these parts. We named it the Gold Eagle because the original house to which we have added was the home of William Henry DeSaussure who was the first (1794) successfully to coin a gold eagle when he was Director of the Mint.

I am financing a very interesting engineering job in the building of a causeway to connect St. Helena and Dataw Islands where they are a mile apart. We have 33 to 35 of our colored neighbors working on it and the pay roll runs $200 a week. It is amazing the speed with which the fascines are bundled and laid and the mud and sand crusted over them. I bought a sand sucker but we are not using it, the men with the barrows and planks are going so much faster. They are the happiest lot to have that steady job under H. Reeves Sams who was born on Dataw and whose forefathers for so many generations owned it. They always referred to their helpers as "servants," never as "slaves." Since he has taken charge, the old people who were born on Dataw are flocking back so as to be near him.

And don't you think that $200 a week for 33 men is not a lot of money. Coming just now when taxes are due, it is letting many of them save their homes.

You would wonder how such a glorious country could be so poor. In my opinion it is all due to the civil war and the boll weevil. I can show you here the very room where in Beaufort's day of wealth and arrogance 70 years ago, the secession of South Carolina was signed. But Edward Rhett, one of Beaufort's leaders, opposed the selection of Jefferson Davis. So when Beauregard wanted to attend to the fortifications, Davis told him to concentrate on Charleston and Savannah. When the Union Navy got here in October '61, years ahead of the capture of any other Southern port, Davis made no effort to dislodge the Union troops who successfully managed from here, and it made the handiest sort of a base for Sherman's March to the Sea. Then in August '64, all the big plantations were cut up into 5 and 10 acre tracts and sold to the colored people. And the colored

people had the mistaken idea that freedom included forgetting how to work as well as learning how to read. But the long fibre Sea Island cotton kept them going until the advent of the boll weevil in 1918. Regular lines of ocean going steamers came into Port Royal and now to see that place gives me the Willies.

REINFORCED CONCRETE COLUMN TEST FUND

Cash, Material donations and services of assistants previously acknowledged (See October JOURNAL)...	$22,384.53
Additional cash contributions since publication of Oct. JOURNAL:	
Abrams, Duff A., New York City.....	100.00
Dixon, D. H., New York City	100.00
McMillan, F. R., Chicago..	100.00
Turner, H. C., New York City	250.00
Upson, M. M., New York City	125.00
Young, R. B., Toronto	100.00
Rail Steel Bar Association, Chicago.....	1,000.00
	$24,159.53

BUILDING UNITS OF STANDARD(?) SIZES

IF MR. WILK'S letter on page 13 of the November *News Letter* almost failed to make its point, it is because of an unfortunate typographical error. In the eighth line, "building tile were set at 5½ in. high," should have said "5 in. high." in order to make obvious the just complaint (if complaint it be) that standard sizes do not mean much.

A PETITION TO AMEND BY-LAWS

THE UNDERSIGNED members of the American Concrete Institute, acting in conformity with Section 1 of Article 6 of the By-Laws of the American Concrete Institute, respectfully petition for the amendment of Section 3, Article 2 of these By-Laws with respect to the election of a Nominating Committee.

This section now reads:

"Section 3—There shall be a Committee of five members on nomination of officers elected by letter ballot of the members of the Institute which is to be canvassed by the Board of Direction at least six months prior to the annual convention."

We would amend this section by the addition of the following sentence:

"The President, Vice-President, Secretary, Treasurer and the Directors from the six geographical districts shall be ineligible for membership on this Committee."

D. A. Abrams	Inge Lyse
W. P. Anderson	F. R. McMillan
E. D. Boyer	J. C. Pearson
R. L. Brown	R. S. Phillips
Cloyd M. Chapman	Howard Rhode
R. B. Crepps	F. E. Richart
H. E. Davis	H. A. Sawyer
Raymond E. Davis	L. C. Shuman
P. J. Freeman	W. A. Slater
H. F. Gonnerman	G. E. Troxell
S. C. Hollister	H. C. Turner
F. H. Jackson	M. M. Upson
A. E. Lindau	R. B. Young
Arthur R. Lord	

Plan to be in Milwaukee Feb. 24-26

IN CONSTANT REPAIR

(A Concersation with Dr. Sam'l Johnson)

Dr. Johnson: A man, Sir, should keep his Friendships in a state of Constant Repair.

The Author: And no less, Sir, his Information.

11—UNIFORMITY

BY ARTHUR R. LORD

WHEN the Member from Russia* had in the most obliging way, reduced my bag of questions to the point where I had to consider carefully their diplomacy, he suddenly came back at me with a questionnaire that, numbered from one almost to infinity, covered rather thoroughly the fundamentals of concrete making and testing. You may readily perceive the possibilities. Here is Russia, returned as a Rip van Winkle, rubbing his eyes and looking over this American setting. Quite naturally he wishes to possess himself of the essentials of our methods, with the useless vestiges of their evolution stripped away. My brother-in-law took a worn shirt to his Chinese tailor that a new supply might be made. When the order was filled every new shirt had been torn and neatly patched in faithful copy of the model! Let Russia make her own original mistakes, as she probably will, instead of copying the rips that we have gathered in the bramble patch of experience.

Faced with such a set of questions what answer should I have made? I am tempted to set some of these matters down, that I may be corrected if I mislead him.

He wished to know of me why Slater reported the relation between 7-day and 28-day cylinder strength as $S_{28} = S_7 + 30\sqrt{S_7}$ when Russian tests clearly prove that it is $S_{28} = S_7 + 7\sqrt{S_7}$—obviously a much easier formula to remember. I pass this on to the new Chairman of the Joint Committee. My Russian friend wished to know of me just what use should be made of "fineness modulus" and the "slump" test. That I commend to Duff Abrams to ponder. And why does the A. S. T. M. require a tensile test of cement to judge of its fitness for use in concrete, for which a compressive test is specified? For that, at least, Duff is not responsible. And so on, to considerable length. I judge that in Russia, no less than in South Africa, they have tried out other people's ideas; had "surprising results."

I gave one insistent answer to all these questions. *Uniformity* is the key note, the basic requirement of successful concrete. The slump test, fineness modulus, Slater's formula, briquette strength—all these mean little in themselves in comparing results through any considerable range of conditions. They are most useful tools for checking and maintaining the necessary final uniformity under controlled conditions. So far as I know, the compressive strength of concrete cannot be determined from the tensile strength of a cement briquette. All that can be had from briquettes is a comparison of one particular cement with our past experience with cements generally. And that may be done as well by briquettes as by cylinders. The same fineness modulus may be had with a wide range of grading in particle sizes. But with a single source of material, processed in the same plant, a variation in the fineness modulus reveals a change in grading that will cause trouble at the mixer. In deter-

*See "New Russia, Second Hand," Nov. 1930, *News Letter*, p. 11.

mining the surface moisture in aggregates absolute accuracy is less important than dependably uniform repetitions.

A proper self esteem, as an engineer, leads me to believe—I even claim to have demonstrated this—that most of our concrete difficulties would be eliminated, if we could control the material producers on our left hand and the field construction forces on our right, making both of them do their work systematically and uniformly. Given uniform cement, sand, and gravel or stone, measured accurately, mixed with the same amount of water and for the same time; and every batch of concrete would be like every other batch—as good or as bad as you deserve, in accordance with your recipe. Convey and place this uniform concrete in a uniform manner; cure it faithfully in all its parts, and the job may readily be made just about perfect. That's equally simple in Russia or America. And it really is far more simple than most people realize, although it certainly does require a constantly maintained effort and attention—a will to succeed. We shall all come to uniformity eventually, why not start now?

CONCRETE IN BUILDING CONSTRUCTION

THE ENGINEERS CLUB of Philadelphia announces a special concrete program a week ahead of the American Concrete Institute convention. The meeting is to be held Tuesday, February 17, when the subject will be "Concrete in Building Construction." There will be two sessions, afternoon and evening, with dinner intervening; the meeting and dinner to be held there at the Engineers Club. At the afternoon session, the subjects will be: "Concrete Masonry Back-Up," "Decorative Concrete," "Influence of Concrete on Architectural Style," "Inspection and Testing of Concrete Structures and Materials," "Lightweight Aggregate"; in the evening, "Tall Buildings," "Rigid Frames," "Floor Finishes." The Committee in charge is composed of F. O. Dufour, vice-president, United Engineers and Constructors; P. M. Taylor, consulting engineer; W. H. Gravell, consulting engineer, T. P. Watson of the Pennsylvania Railroad and W. E. McComas District Engineer, Portland Cement Association.

A. C. I. MEMBERS

CHARLES R. GOW

now postmaster of Boston and Professor of Humanics at the Massachusetts Institute of Technology, is the author of "Foundations for Human Engineering" (The Macmillan Co.), whose reviewer in *Engineering News Record* says: "The many engineers who are fortunate enough to know Mr. Gow will agree that he was just the man to fill a chair of humanics at an engineering school; and those not so fortunate will join the others on reading this book." Colonel Gow was a director of the Institute 1916-19; vice-president 1920-21; winner of the second Wason medal for the presentation of "the most meritorious paper" at the 1917 convention: "History and Present Status of the Concrete Pile Industry."

FRANCISCO GOMEZ-PEREZ

of Mexico City, who represented the American Concrete Institute at the eighth National Convention of Mexi-

can Engineers, at Durango, Oct. 11-16, presented to the convention a "Message of Greeting," from the Institute and a paper: "Measurement of Temperatures in Concrete While Hardening." The convention adopted a resolution of congratulation to the author of the paper and graciously suggested that technical societies in Mexico and certain government bureaus would find it advantageous to hold membership in the American Concrete Institute.

CARROLL A. BULLEN

who, as a pinch-hitter, discussed "cementing oil wells," when the scheduled speaker did not appear at the New Orleans convention of the Institute last February, is now chief engineer for Dolese Brothers Co., producers of crushed stone and washed sand, 13 West 13th St., Oklahoma City. He was formerly with the Lehigh Portland Cement Co., at Chicago.

RALPH J. FOGG

who left Lehigh University, where he was Professor in charge of Department of Civil Engineering, for consulting practice, is now at 11 Park Place, New York, of the firm, Condron, Post & Fogg, associates, supplementing the engineering practice of Condron & Post, Chicago.

DEMORALIZING

AN ENGINEER of wide repute and large accomplishment is reported to have made severe criticism of the Institute recently: "He stated that the JOURNAL was quite demoralizing. It contained so much of interest to him that he found he was giving it far more attention than he should in comparison with the other matters on hand."

NEW MEMBERS

FOURTEEN applicants for membership in October have been approved by the Board of Direction as follows:

Alcoa Power Co., Ltd., Arvida, Que., Canada (C. P. Dunn, Ch. Engr.)

Anderson, Emanuel, American Gas & Electric Co., 30 Church St., New York, N. Y.

Bergford, L. M., 556 Builders Exchange, Minneapolis, Minn.

Beretta Engineers, Inc., J. W., 1203 Nat'l Bank of Commerce Bldg., San Antonio, Texas (J. W. Beretta, Pres.)

Besser Manufacturing Co., Alpena, Mich. (J. H. Besser)

Cinder Block & Material Co., P. O. Box 137, Cedar Rapids, Ia. (H. L. Spaight)

Gillon, Gerald Norton, 12 Manilla St., Dannevirki, New Zealand

Joffe & Co., A. S., P. O. Box 4147, Johannesburg, S. Africa (A. S. Joffe)

Marin, Joseph, Dept. of Civil Engineering, Rutgers University, New Brunswick, N. J.

Martin, F. J., 315 Avenue Bldg., Saskatoon, Sask., Canada

Mosher, Clarence H., Clark Ave., Cornwall-on-Hudson, N. Y.

Neil, J. S., 744 Crescent Road, Calgary, Alta, Canada.

Roth, Jr., Charles Oliver, 206 E. Gravers Lane, Chestnut Hill, Phila., Pa.

Stanford University, Stanford University, Calif. (Nathan Van Patten, Director)

Papers and Reports—In this Issue

Proceedings American Concrete Institute

Vol. 27—Pages 317-384

STRENGTH AND SHRINKAGE OF MORTARS MADE WITH BLENDS OF PORTLAND CEMENT AND POZZUOLANIC MATERIALS 317
By C. A. HUGHES and A. S. LEVENS

TESTS OF BONDING FLOOR FINISH TO SLABS OF HAYDITE AND GRAVEL CONCRETE 339
By F. E. RICHART and V. P. JENSEN

DEFLECTION OF REINFORCED CONCRETE MEMBERS 351
Report of Committee 307, T. D. MYLREA, Author-Chairman

CONTINUOUS BEAMS AND FRAMES IN BUILDING CONSTRUCTION 359
By T. U. BERG

DISCUSSION OF "COMPRESSIVE STRENGTH OF CONCRETE IN FLEXURE—AS DETERMINED FROM TESTS OF REINFORCED BEAMS" 377

Publication of discussion of the contributions in the following pages is on a time schedule. Please see inside cover pages.

Strength and Shrinkage of Mortars Made with Blends of Portland Cement and Pozzuolanic Materials

C. A. HUGHES* AND A. S. LEVENS†

A POZZUOLANIC material has been defined[1] as "a substance, while not necessarily cementitious by itself, possesses constituents which will combine with hydrated lime at ordinary temperatures in the presence of moisture to form stable insoluble compounds of cementitious value. When added to lime mortar in addition to or in partial substitution for the sand, it will impart hydraulic properties and greater strength. When added to concretes and mortars made with portland and similar cements by the consequent removal of the free lime, it will impart to the material an increased degree of resistance to various agencies which are normally liable to cause disintegration."

If it be accepted[2] that the compounds formed by a pozzuolana and the lime hydroxide liberated during the hydration of Portland cement are of cementitious value, then the use of pozzuolanas with Portland cement will affect the water-cement ratio strength relations.

It is claimed[3] also that pozzuolanic admixtures reduce defects caused by shrinkage, particularly crazing and checking.

*Assistant Professor of Structural Engineering, University of Minnesota.
†Assistant Professor of Drawing and Descriptive Geometry, University of Minnesota.

1"*Lime and Lime Mortars*". A. S. Cowper, Special Report No. 9, Department of Scientific and Industrial Research. London.

2"Tufa Cement, as Manufactured and Used on the Los Angeles Aqueduct"—J. P. Lippincott, *Transactions of American Society of Civil Engineers*. Vol. 76, 1913, page 520. This paper and discussion containing much data from tests made during the construction of the Los Angeles Aqueduct and elsewhere is to the authors' knowledge, the most complete treatment of blended cements as used in America. Mr. Lippincott's conclusions are as follows:

3An American Pozzuolana and Its Effect on Portland cement Concrete. Dr. E. Lee Heidenreich, *Rock Products*. Sept. 29 and Oct. 27, 1928.

1st.—The tufa, when finely ground with cement and used in concrete, combines both chemically and mechanically. Blends of 50%, when mixed with sand, give greater tensile strength after 10 days than straight cement mixed with the same proportion of sand. The leaner the mixture, the greater the relative superiority of the tufa cement. In compression, the tufa cement concrete is less strong (20%) in rich mixtures (1:2:4), and as strong in leaner mixtures (1:3:6). (footnote cont'd p. 318)

The tests for which the data are presented herewith were undertaken to determine the effect of four pozzuolanic materials on the strength and shrinkage of mortars. Compression tests were made on 218-2x4 in. cylinders and shrinkage measurements taken on 27-2x2x10 in. beams.

MATERIALS

Cement. Two brands of portland cement were used. These are designated by the letters N and A. The N brand was used for the majority of the tests. Both cements complied with the requirements of the A. S. T. M. specifications.

Fine Aggregate. The fine aggregate was a local concrete sand graded from the No. 4 sieve down with a fineness modulus of 2.6. It was used in a bin-dry condition and had a net absorption of 0.6 per cent. Care was taken to maintain a uniform grading for all mixes.

Water. The mixing water was from the Minneapolis City supply and was at 70° F. when used.

Pozzuolanic Materials.

F—a volcanic ash obtained from a deposit near Fresno, Calif.

P—a volcanic ash sold commercially as a concrete admixture.

S—Blast furnace slag used as a raw material in the manufacture of portland cement.

H—Burnt shale sold commercially for concrete aggregate.

F and P were used as received. S and H were ground in a ball mill. No measure of the fineness of the pozzuolanas was obtained other than that all passed the No. 200 sieve, which is of little significance since pozzuolanic action results from the very fine particles.

The pozzuolanas were substituted for cement by weight so that the total weight of cement or cement plus pozzuolana was constant for all mixes. The cement and pozzuolana were blended to a uniform color before mixing the mortars.

2nd.—Tufa cements, in tension, of blends from 30 per cent to 80 per cent show a continued growth in strength with age, as far as tested, up to 5 years, and in this respect are superior to straight cements, which usually show declining strengths.

3rd.—The tufa concretes must be handled with greater care with reference to both cold and drying, and forms should be left in place about one-third longer. In massive work this feature is negligible.

4th.—From the fact that the tufa cement is more finely ground and, in part, combines mechanically with other aggregates, carrying the graduation of fineness one step farther, it makes a denser and more impervious concrete.

5th.—Where cements are high priced, a substantial economy may be effected if deposits of tufa are available. These conditions occur in portions of Western America. The milling cost of producing the extra barrel of tufa cement in small plants should not exceed 75 cents.

Pozzuolanic Activity

The relative activity of the four pozzuolanas was determined by a method used by the Building Research Board.[1] This consisted of adding 0.6 g. of pozzuolana to 0.4 g. of slaked lime in a test tube, covering with pure water and shaking every twelve hours.

The interaction between the solution of lime in water and the pozzuolana will give rise to hydrated calcium alumino-silicates of complex or indefinite chemical structure; these hydrates will be formed in a colloidal condition, and are much more bulky than the pozzuolana or lime, with the result that after seven days the tube will contain an increased volume of solid matter as measured by the height it extends up the tube, also the rate of settlement, owing to its flocculent nature will be slower; the activity of the pozzuolana can, therefore, be judged by inspecting the tubes at different times after shaking. In comparison, the volume of the solid matter immediately after mixing will be much smaller and it will settle immediately.[1]

The per cent increases in volume following a 30 minute settling period are shown in Table 1. Also see Fig. 1.

TABLE 1—PER CENT INCREASES IN VOLUME RESULTING FROM POZZUOLANIC ACTIVITY

Age Days	Pozzuolana			
	F	P	S	H
2	8	64	16	52
3	46	79	236	73
4	92	89	245	83
5	123	97	245	94
6	143	100	247	115
7	146	100	254	135

Mixing. From each batch consisting of 1500 grams of sand and 500 grams of cement, or cement plus pozzuolana, were made three 2x4 in. cylinders or one beam. The method of mixing the mortars was that specified by the Standard Specification for Portland cement. Four sets of cylinders and two sets of beams were made, each set being molded on separate days.

Consistency. All mixes were brought to a "just plastic" condition. The quantity of water required for each mix was determined from preliminary trial batches and checked on the mixes used in making the specimens. In the trial batches, water

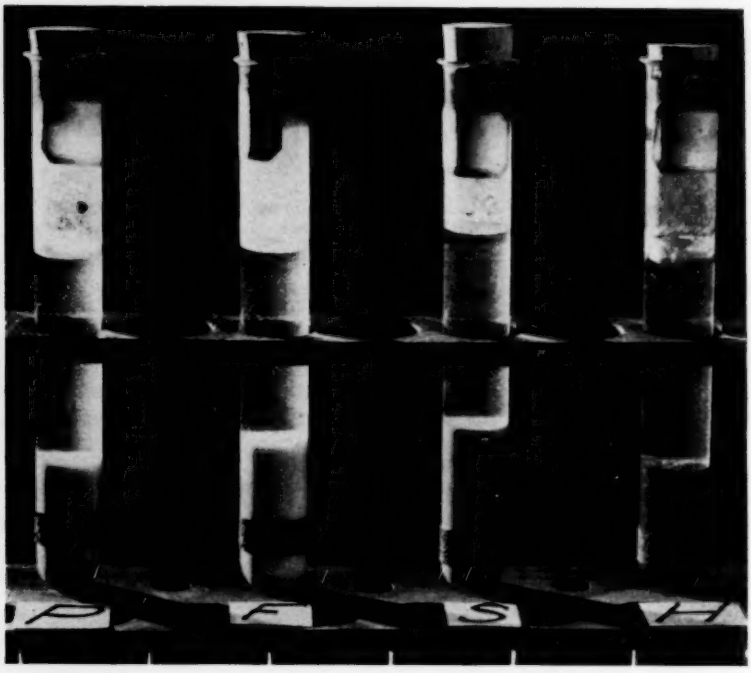


FIG. 1—TEST OF POZZUOLANIC ACTIVITY

The top edges of the black tapes at the bottoms of the test tubes are at the original levels of the mixtures of pozzuolanic materials and lime.

was added in increments until the mix changed from a more or less granular to a plastic, cohesive state. This change is well defined for most mortars. The quantities of water determined by independent trials checked closely. The mixes when "just plastic" were considered to be of equal consistency.

Curing. All specimens were placed in a moist closet immediately after molding and stripped at 20 to 22 hrs. All compression specimens were then placed in water storage at 70°. The subsequent curing is recorded in Table 2. The curing conditions for the shrinkage specimens following stripping is given in Table 4.

Testing. All specimens were tested in the moist condition, the air-cured specimens being submerged in water for 24 hours im-

TABLE 2—COMPRESSIVE TESTS.

Specimens—2 x 4 in. cylinders.
Mix—1:3 by weight.
Pozzuolanas substituted for cement and blended with it before mixing mortars.
Water used in calculating the ratios of water to cement and water to cement plus pozzuolana corrected for the absorption of the sand only. The absorption of the pozzuolana is not considered.
All strengths are an average of four specimens made on different days, except as noted.
All specimens submerged 24 hours prior to testing.

Pozzuolana	Water-Cement Ratio (Vols.)	Ratio—Water to Cement Plus Pozzuolana (Wt.)	Abrams Water Factor for Admixture	Compressive Strength Lb. per Sq. In.			Strength Ratios Based on Plain Mixes			Curing
				7 Day	28 Day	90 Day	7 Day	28 Day	90 Day	
N Cement—10 Per Cent Substitution by Weight.										
None	0.66	0.437	10	2620[2]	4060	5480	100	100	100	1 day moist
F	0.76	0.457	10	2220	3880	4585	85	96	84	room
P	0.82	0.492	45	1730	3565	4840	66	88	89	Remainder
S	0.75	0.452	5	2030	4330	5600	77	107	102	under water
H	0.77	0.462	15	2215	4230	6045	85	104	110	at 70°.
A Cement—10 Per Cent Substitution by Weight.										
None	0.63	0.417	10	3790	5640	6560	100	100	100	
F	0.74	0.442	15	2820	4600	5020	74	82	77	
P	0.79	0.472	45	2805	4320	5610	74	77	85	ditto
S	0.73	0.437	10	3075	5010	5275	81	89	80	
H	0.74	0.442	15	3210	5010	6790	85	89	103	
N Cement—30 Per Cent Substitution by Weight.										
None	0.66	0.437	10	2620[2]	4060	5480	100	100	100	
F	1.05	0.482	5	1230	2610	4430	47	64	81	
P	1.25	0.592	41	925	2100	4615	35	52	84	ditto
S	0.98	0.457	—3	1830	4230	6405	70	104	117	
H	0.93	0.432	—8	1785	3490	6390	68	86	116	
None	Ditto			2620[2]	3010	3030	100	100	100	1 day moist
F				1265	1740	1940	48	58	64	room, 6 days un-
P				840	1260	1320	32	42	44	der water at 70°
S				1715	2670	2870	65	89	95	remainder in air
H				1745	2570	2990	67	87	99	of Laboratory.[1]

[1]Average of 3 specimens.
[2]Average of 8 specimens.

mediately prior to testing. One specimen from each batch was tested at each of the ages of 7, 28 and 90 days.

Shrinkage Measurements. The shrinkage measurements were taken with a Berry strain gauge on an 8-in. gauge length.

Initial readings were taken on the shrinkage specimens at 20 hr. for the first or "a" set and at 22 hr. on the second or "b" set. Drilled and countersunk brass plugs were used for gauge points. Three standards and all specimens of one set were read with one setting of the dial. Two such series of readings were taken each time, the dial being shifted between series. Third readings were taken when necessary. The relative movements were then calculated from the averages of those differences not varying by more than four "points" which corresponds to a movement of 0.001 per cent.

The per cent linear changes relative to the 1 day (20 and 22 hr.) lengths and the differences between the "a" and "b" specimens are given in Table 3 for 5 ages. The differences shown vary from a maximum of 0.0128 to a minimum of 0 and have an average value of 0.0033 per cent relative movement. This agreement is considered satisfactory, particularly in view of the fact that the "a" and "b" sets were made one week apart.

DISCUSSION OF RESULTS

In Table 2 the quantity of effective water is expressed both as a ratio of the quantity of portland cement by bulk volume, and as a ratio by weight of the blended cement. The quantity of effective water* used in each batch is easily obtained by multiplying the latter ratio by 500. With one exception (the mix containing 30 per cent substitution of H) more water was required for the blended mixes than for the plain mixes. Except for the mixes containing P, the additional quantities of water required were small, but since the quantities of portland cement were reduced, the differences in water-cement ratio are considerable.

Abram's "water factor"[4] defined as "the quantity of mixing water required by the addition of a unit weight of any constituent in order to maintain a given workability," is given for each

*The term effective water is used here to mean the quantity of added water corrected for the absorption of the aggregate, no allowance being made for absorption by the pozzuolanas.

[4]Duff A. Abrams, "Tests of Powdered Admixtures in Concrete." Proc. A. S. T. M., Vol. 29, 1929, Part II, p. 627.

material in the fourth column of Table 2. These were calculated on an assumed water-factor for cement of 10 per cent. Taking into account the fact that small changes in the total quantity of water result in very large changes in the water factor, the agreement between the values obtained for the blends of the two cements and the two percentages of each pozzuolana is good.

The authors believe that the accuracy of the method used in determining the quantity of water may be judged by comparison of the cement factors for the same pozzuolana obtained from different mixes. The ratios of water to cement plus pozzuolana are given to three decimal places for convenience in checking since the use of these figures when rounded to two places would give cement factors appreciably different from those shown.

Compressive Strength

The relative strengths, plotted from the data of Table 2 are shown in Fig. 2. All four blends show a considerable loss in strength at 7 days for both conditions of curing. In general, the compressive strengths of the blended mortars increase relative to those of the plain mortars with increase in age. Materials S and P, showing the greatest and least pozzuolanic activity respectively, also had the greatest and least relative strengths.

Moist cured specimens. With the N cement, the use of both 10 and 30 per cent substitutions of S and H gave relative strengths at 90 days greater than 100. With both N and A cements the 10 per cent substitution of F and S resulted in some retrogression at 90 days from the 28 day relative strengths. The average relative strength of the 10 per cent blended specimens made with N cement were about equal at 7 days and considerably higher at 28 days than that of those made with A cement.

In general, there is considerable difference in the data given by the two brands. Assuming that there is chemical action due to the pozzuolana, it is natural to expect different effects with different brands of portland cement, as has been found to be the case with other chemically active materials such as calcium chloride.[5]

Air cured specimens. These specimens were moist cured for the first seven days and then left in air. While the small specimens

[5] "Calcium Chloride as an Admixture in Concrete." Bull. 13, Structural Materials Research Laboratory. Lewis Institute. See Fig. 11.

TABLE 3—PER CENT CHANGE IN LENGTH AND VARIATION BETWEEN "a" AND "b" SPECIMENS

Age of Specimen Days	7	142	156	201	339
Specimen	N Cement—10 Per Cent Substitution				
Na	—0.0025	+0.0982	+0.0627	+0.1140	+0.1285
Nb	—0.0057	+0.0945	+0.0615	+0.1120	
diff.	0.0032	0.0037	0.0012	0.0020	
N1Fa	—0.0057	+0.1012	+0.0662	+0.1140	+0.1280
N1Fb	—0.0060	+0.1055	+0.0667	+0.1182	+0.1335
diff.	0.0003	0.0043	0.0005	0.0042	0.0055
N1Pa	—0.0007	+0.1145	+0.0782	+0.1245	+0.1415
N1Pb	—0.0042	+0.1165	+0.0772	+0.1262	+0.1450
diff.	0.0035	0.0020	0.0010	0.0017	0.0035
N1Sa	—0.0035	+0.1152	+0.0772	+0.1250	+0.1420
N1Sb	—0.0047	+0.1117	+0.0710	+0.1220	+0.1392
diff.	0.0012	0.0035	0.0062	0.0030	0.0028
N1Ha	—0.0057	+0.0982	+0.0627	+0.1100	+0.1262
N1Hb	—0.0060	+0.1060	+0.0660	+0.1150	+0.1335
diff.	0.0003	0.0078	0.0033	0.0050	0.0073
Maximum diff. —0.0078	Minimum diff. —0.0003		Average diff. —0.0032		
	A Cement—10 Per Cent Substitution				
Aa	—0.0012	+0.0817	+0.0440	+0.0870	+0.1062
Ab	—0.0027	+0.0847	+0.0445	+0.0907	+0.1097
diff.	0.0015	0.0030	0.0005	0.0037	0.0035
A1Fa	—0.0007	+0.0875	+0.0522	+0.0960	+0.1140
A1Fb	—0.0060	+0.0857	+0.0467	+0.0952	+0.1137
diff.	0.0053	0.0018	0.0055	0.0008	0.0003
A1Pa	—0.0020	+0.0917	+0.0557	+0.1005	+0.1185
A1Pb	—0.0010	+0.0905	+0.0532	+0.1012	+0.1185
diff.	0.0010	0.0012	0.0025	0.0007	0.0000
A1Sa	—0.0005	+0.0920	+0.0530	+0.1005	+0.1165
A1Sb	—0.0022	+0.0900	+0.0502	+0.0967	+0.1122
diff.	0.C017	0.0020	0.0028	0.0038	p.0043
A1Ha	+0.0007	+0.0810	+0.0455	+0.0907	+0.1055
A1Hb	—0.0040	+0.0787	+0.0420	+0.0885	+0.1035
diff.	0.0047	0.0023	0.0035	0.0022	0.0020
Maximum diff.—0.0055	Minimum diff.—0		Average diff.—0.0024		

TABLE 3—CONTINUED

Age of Specimen Days	7	142	156	201	339
Specimen	N Cement—30 Per Cent Substitution				
N3Fa	—0.0007	+0.1080	+0.0790	+0.1232	+0.1382
N3Fb	—0.0015	+0.1202	+0.0860	+0.1317	+0.1485
diff.	0.0008	0.0128	0.0070	0.0085	0.0103
N3Pa					
N3Pb	+0.0005	+0.1377	+0.1030	+0.1542	+0.1642
diff.					
N3Sa	—0.0017	0.1392	+0.0962	+0.1460	+0.1637
N3Sb	—0.0062	+0.1360	+0.0920	+0.1452	+0.1592
diff.	0.0045	+0.0032	0.0042	0.0008	0.0045
N3Ha	—0.0035	+0.0920	+0.0584	+0.0967	+0.1165
N3Hb	—0.0042	+0.0945	+0.0602	+0.1015	+0.1220
diff.	0.0007	0.0025	0.0018	0.0048	0.0055

Maximum diff.—0.0128 Minimum diff.—0.0007 Average diff.—0.0053
Grand average difference—0.0033

undoubtedly dried out quickly, this curing was thought to be comparable to that obtained in ordinary construction. If the drying were more rapid, the initial curing was probably more efficient than that of the usual job. For this curing condition, the average strengths of the specimens made with the S and H materials substituted for 30 per cent of the cement by weight were 66, 87 and 97 per cent of the plain mortar strength at 7, 28 and 90 days respectively, whereas the corresponding figures for F and P were 40, 50 and 54.

Since the rate of the reaction between the silica and alumina of the pozzuolanas and the lime of the cement is low, it is to be expected that strength comparisons would be more favorable for the blended cements for long periods of moist curing. This is indicated by the slope of the graphs of Fig. 2, particularly by those for the 30 per cent blends. It is significant that pozzuolanic cements and blends of portland cement and pozzuolanic materials have been used almost exclusively for hydraulic structures

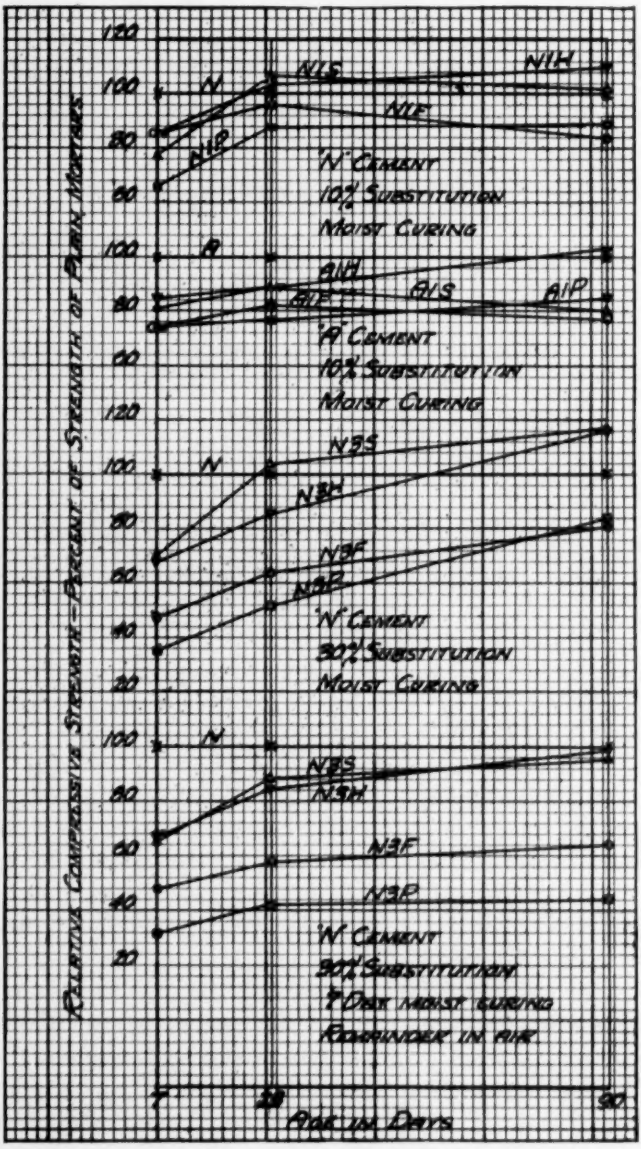


FIG. 2—COMPRESSIVE STRENGTHS OF 1:3 MORTARS MADE WITH BLENDED CEMENTS RELATIVE TO PLAIN MORTARS

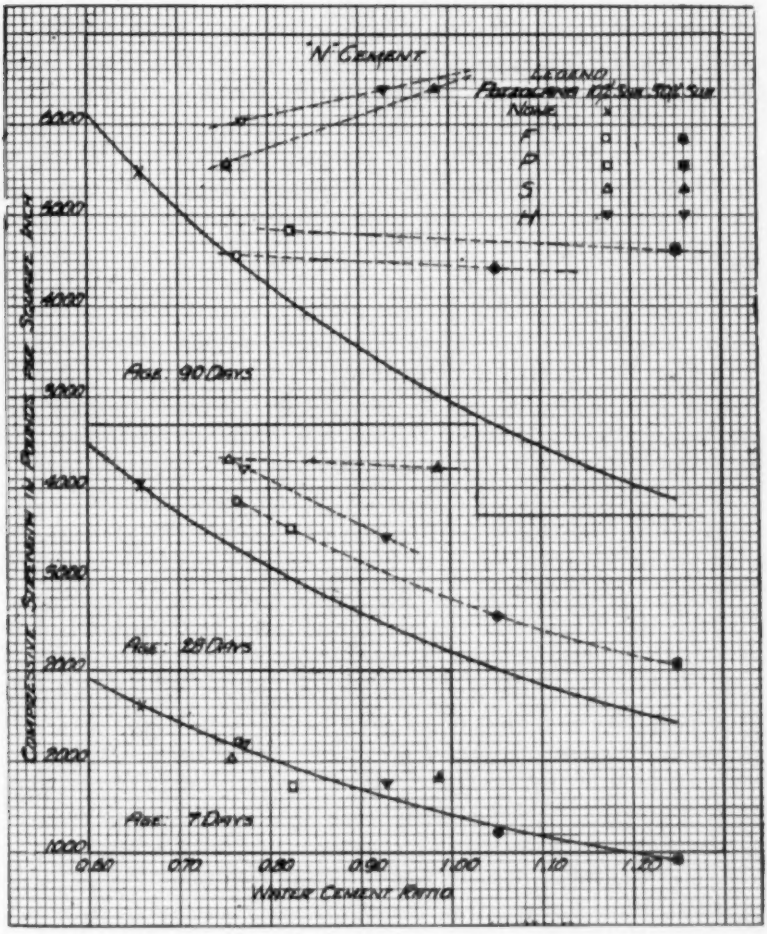


FIG. 3—WATER-CEMENT RATIO STRENGTH RELATIONS FOR BLENDS OF POZZUOLANAS WITH "N" CEMENT

(tunnel linings, aqueducts, sea-walls and the like) in which the conditions are favorable for continued moist curing.[2]

It is at once evident from Fig. 2 that admixtures S and H are more desirable than F and P, particularly for the shorter periods of moist curing and for air curing following a short period of moist curing. When used with N cement, the 30 per cent substitution resulted in greater reductions in strength at early ages, but for continuous moist curing gave strengths at 90 days equal to or greater than those for 10 per cent substitution. The fact that blends of 30 per cent of S and H gave strengths 17 per cent and 16 per cent greater respectively than plain cement is particularly notable.

Water-cement Ratio Relations

In Figs. 3 and 4 the water-cement ratio strength relations are shown for the moist cured specimens. The dotted lines of Fig. 3 joining the points for the 10 and 30 per cent blends have no special significance, but are shown only as an aid in reading the diagrams. The solid lines are believed to approximate the relations for plain mixes. The data for the solid lines were obtained in the following manner: The 7 day strengths were plotted on semi-log paper and a straight line drawn through them. The ordinates for the 28 day and 90 day curves were then obtained by multiplying the 7 day ordinates by the ratios of the 28 day and 90 day to 7 day strengths of the plain specimens. These ratios were 1.55 and 2.09 for N cement and 1.49 and 1.73 for the A cement. Corresponding ratios given by Gilkey[6] for the average portland cement are 1.52 and 1.94. This procedure assumes that the pozzuolanic materials have caused no increase in strength at 7 days over plain specimens made with equal water-cement ratios. If the blended specimens were stronger than corresponding plain specimens (equal water-cement ratios) at 7 days as is likely (see Fig. 5), then the solid lines should slope to the right at a greater angle and the pozzuolanic effect would be greater than that indicated in Figs. 3 and 4.

In general, the divergence from the solid lines increases with increase of age and per cent of substitution. The 10 per cent

[6] "A Method for Predicting Concrete Strengths with Increased Precision." Proc. A. C. I., Vol. XXIV, 1928.

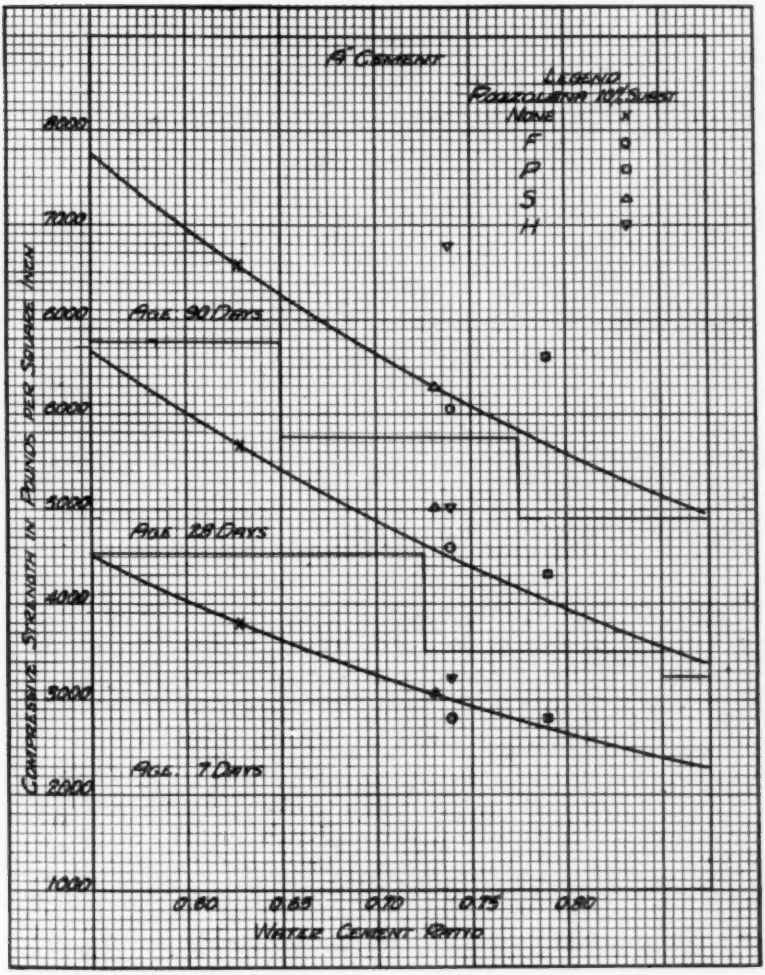


FIG. 4—WATER-CEMENT RATIO STRENGTH RELATIONS FOR BLENDS OF POZZUOLANAS WITH "A" CEMENT

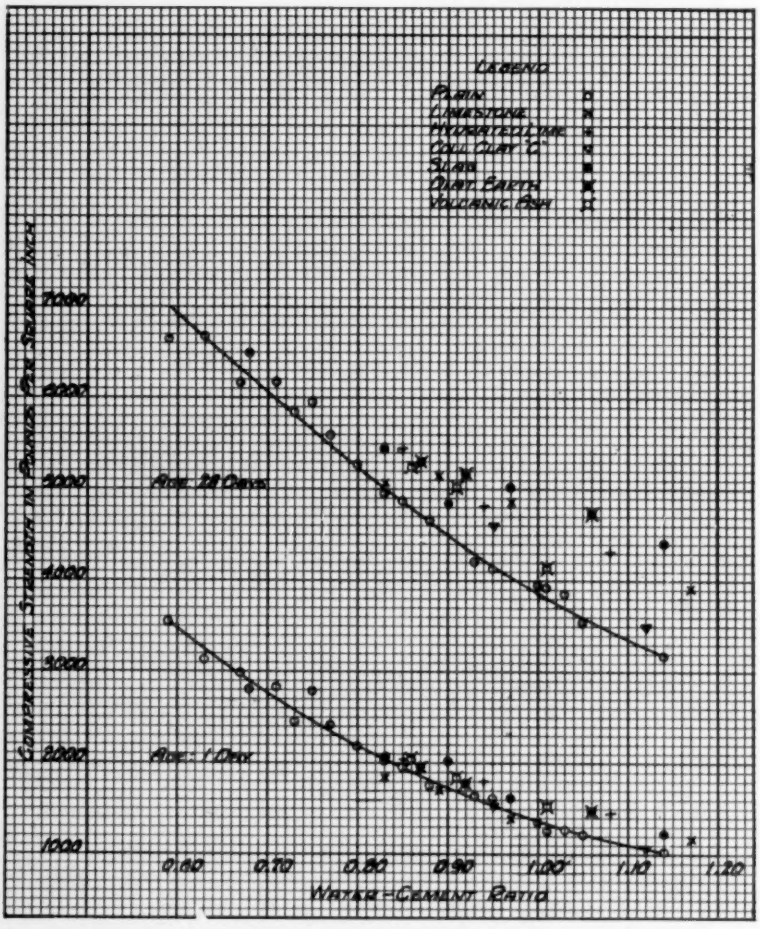


FIG. 5—WATER-CEMENT RATIO STRENGTH RELATIONS FOR CONCRETES CONTAINING VARIOUS ADMIXTURES. DATA FROM TABLES II AND III OF "TESTS OF POWDERED ADMIXTURES IN CONCRETE." BY DUFF A. ABRAMS, PROCEEDINGS A. S. T. M., VOL. 29, 1929

blends of F with N cement and F and S with A cement fall on the solid line at 90 days and yet are above it at 28 days. This does not appear to be in line with the data for the other blends. This may be due to the limited number of specimens tested or may be explained in the following manner.

Since the pozzuolanic materials are chemically active, they will affect the hydration of the cement and we have in effect a new cement having strength-age and water-cement ratio strength relations peculiar to itself just as is the case with different brands of portland cement. Regarding the blends in this way, it would not be unreasonable to expect different percentages of pozzuolana to give strength-age curves differing in shape as well as in position. Since the strength-age curves for different brands of portland cement have been shown to cross,[7] the positions of the points for the 10 per cent blends of F and N cement and F and S with A cement at 90 days might be explained analogously.

Fig. 5 was prepared from data in Tables 2 and 3 of "Tests of Powdered Admixtures in Concrete."[4] The general relations of the data are similar to those of Figs. 3 and 4, the majority of the points falling reasonably close to the curve for the plain specimens at 1 day whereas at 28 days the mixes containing admixtures fall well above the corresponding 28 day curve. These data seem to show that the strengths for equal water-cement ratios are affected by any finely divided material, not just those commonly regarded as pozzuolanic. The high positions of the points for mixes containing limestone certainly cannot be explained by pozzuolanic action.

Figs. 3, 4 and 5 might be misinterpreted to indicate that greatly increased strengths are obtained from the use of admixtures. Fig. 2 shows clearly that such is not the case when the comparison is made on the basis of the same consistency, particularly at early ages and for ordinary periods of curing.

SHRINKAGE AND WEIGHT TESTS

Data from these tests are given in Table 4. The linear change-age relations are shown in Figs. 6, 7 and 8. In the legends shown the numbers 1 and 3 signify 10 per cent and 30 per cent substitution of pozzuolana for cement. The significance of the various

[7] "Basic Principles of Concrete Making." F. R. McMillan. See Fig. 11, page 26.

TABLE 4—SHRINKAGE AND WEIGHT TESTS.

Specimens—2 x 2 x 10 in. beams. Gauge length—8 in. Instrument—Berry Strain Gauge Mix—1:3 by weight.
Pozzuolanas substituted for cement and blended with cement prior to mixing mortars.
Water-cement ratios calculated as in Table 2.
Curing Conditions—1 day molds, 1 day to 7 days in moist room, 7 days to 142 days in air of laboratory, 142 days to 156 days under water, 156 to 339 days in air. Initial Reading taken at 1 day.

Pozzuolanas	Water-Cement Ratio (Vols.)	Ratio—Water to Cement + Pozzuolana by Weight	Relative Linear Change in Millionths				Linear Change Ratios				Per Cent Change in Weight				Weight Change Ratios			
			1—7 Days Moist Room	7—142 Days Air	142—156 Days Water	156—201 Days Air	1–7 Days Moist Room	7—142 Days Air	142—156 Days Water	156—201 Days Air	1—7 Days Increase	7—142 Decrease	142—156 Increase	156—201 Decrease	1—7 Days Increase	7—142 Decrease	142—156 Increase	156—201 Decreaseo
N Cement—10 Per Cent Substitution by Weight																		
None	0.66	0.437	41	1004	342	509	100	100	100	100	0.31	4.53	4.49	3.76	100	100	100	100
F	0.76	0.457	59	1093	369	496	144	109	108	97	0.54	5.40	4.87	3.69	175	119	105	98
P	0.82	0.492	25	1180	377	476	61	117	110	93	0.36	5.69	5.29	4.09	116	126	118	109
S	0.75	0.452	41	1176	394	494	100	117	115	97	0.38	4.93	4.68	3.59	123	109	104	96
H	0.77	0.462	59	1080	377	481	144	107	110	94	0.54	5.29	4.89	3.86	175	117	109	103
A Cement—10 Per Cent Substitution by Weight																		
None	0.63	0.417	20	852	390	446	100	100	100	100	0.44	4.07	4.13	3.29	100	100	100	100
F	0.74	0.442	34	899	371	461	170	106	95	103	0.09	4.52	4.46	3.44	20	111	108	105
P	0.79	0.472	15	926	366	463	75	109	94	104	0.42	4.81	4.83	3.85	95	118	117	117
S	0.73	0.437	14	924	394	470	70	108	101	105	0.43	4.30	4.25	3.51	98	106	104	107
H	0.74	0.442	17	814	361	458	85	95	93	103	0.55	4.19	4.20	3.56	125	103	102	108
N Cement—30 Per Cent Substitution by Weight																		
None	0.66	0.437	41	1004	342	509	100	100	100	100	0.31	4.53	4.49	3.76	100	100	100	100
F	1.05	0.482	11	1152	316	450	27	115	92	88	0.24	6.15	5.87	4.24	77	136	131	113
P	1.25	0.592	+5	1372	348	513	—12	136	102	101	0.72	7.96	7.05	5.68	232	176	157	151
S	0.98	0.457	40	1460	435	515	98	145	127	101	0.34	4.97	4.79	3.65	110	110	107	97
H	0.93	0.432	39	971	341	399	95	97	100	78	0.41	5.54	5.13	3.86	132	122	114	103

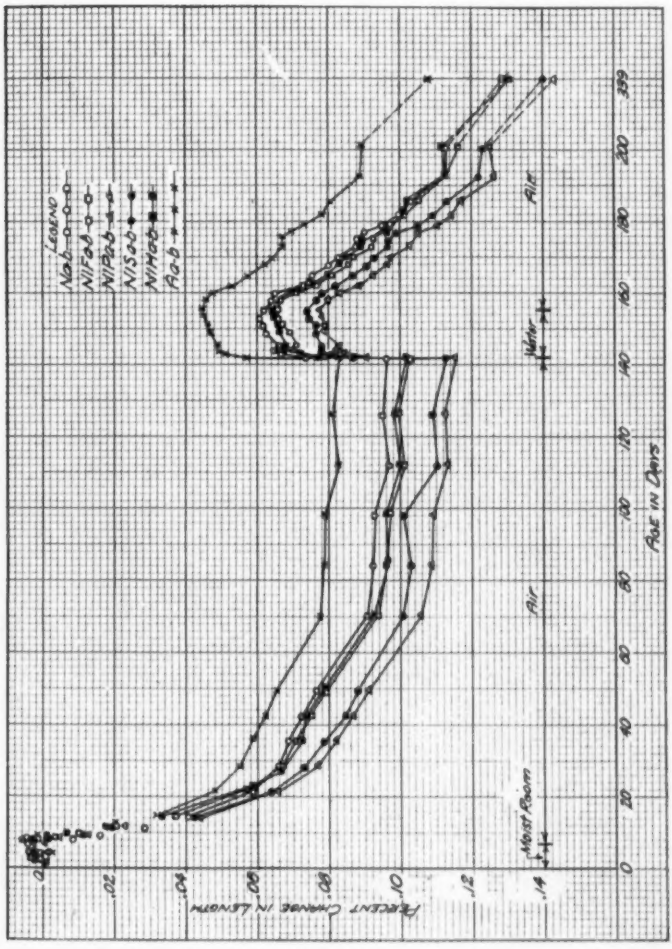


FIG. 6—LINEAR CHANGE GRAPHS FOR 1:3 MORTARS MADE WITH "N" CEMENT AND 10% SUBSTITUTIONS OF POZZUOLANAS.

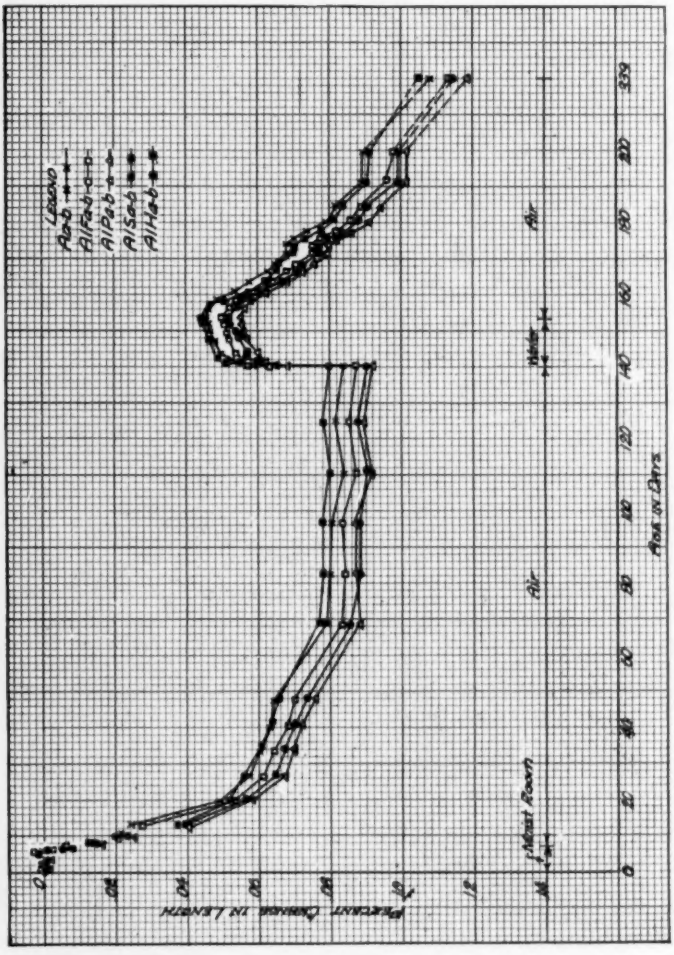


FIG. 7—LINEAR CHANGE GRAPHS FOR 1:3—MORTARS MADE WITH "A" CEMENT AND 10% SUBSTITUTION OF POZZUOLANAS

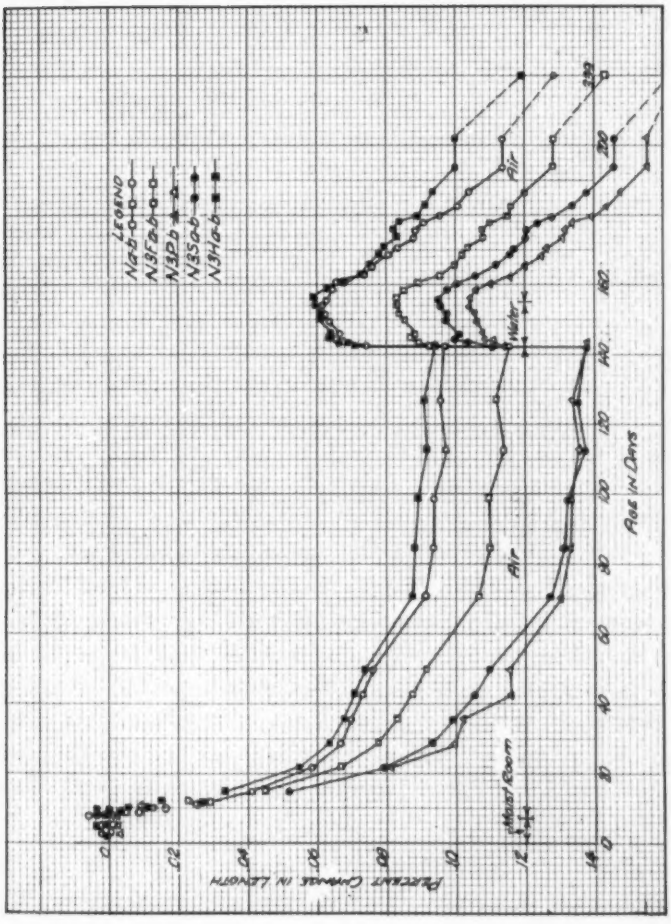


FIG. 8—LINEAR CHANGE GRAPHS FOR 1:3 MORTARS MADE WITH "N" CEMENT AND 30% SUBSTITUTION OF POZZUOLANAS

letters has already been given. The curing conditions for these specimens are given at the head of Table 4 and indicated on the figures. Readings were taken frequently to 201 days. The final reading was taken at 339 days, there being no readings between 201 and 339 days. Weights were taken each time shrinkage measurements were made.

Shrinkage Tests

From Figs. 6, 7 and 8, it will be seen that the shrinkage of the mortars made with blends of H pozzuolana were less for 10 per cent substitution with A cement and 30 per cent substitution with N cement and more for 10 per cent substitution with N cement than that of the corresponding plain mixes. The differences were small, however, and the linear change of these blends may be considered equal to that of the plain mixes. All other blends showed increased shrinkage. The shrinkage curve for the plain mix of A cement is plotted on Fig. 6 to show the effect of the brand of cement. Comparison of Figs. 6 and 7 will show that the difference between the graphs for the two plain mixes is about the same as the maximum difference between the plain and the 10 per cent blended cements. In general, the shrinkage increased with increase in the ratio of effective water to cement plus pozzuolana, the mixes containing S being exceptions to this rule.

The movements of the various mixes relative to those of the corresponding plain mixes are given by the linear change ratios of Table 4. These ratios are less for the shrinkage from 156 to 201 days than from 7 to 142 days particularly for those having 30 per cent substitutions of pozzuolanas. This indicates that with age the volumetric changes may decrease for the blended cements. This is the only evidence given by these tests that shrinkage defects may be less for the blended cements.

Weight Tests

The data from these tests are also given in Table 4. The weight-age graphs (not shown) were similar in shape to the linear change-age graphs. However, the relative positions of the various graphs were different as may be seen from the ratios of Table 4. The order of the weight changes from 7 to 142 days, going from the least to the greatest, is N, N3S, N3H, N3F and

N3P whereas the order of the corresponding volumetric changes is N3H, N, N3F, N3S and N3P.

The increase in weight for the 14 days immersion from 142 to 156 days is the absorption from the air dry condition. It will be seen from Table 4 that the absorptions were greater for the blended cements in all cases.

CONCLUSIONS

While the number of specimens and materials tested is too limited for general conclusions, the following statements appear to be warranted.

For the materials and methods recorded in this report:

1. For equal consistencies, blended cements required more water per unit of cementing material than portland cement (there was one exception, see text).

2. The compressive strengths of mortars made with the blended cements were considerably less at early ages than those of portland cement mortars of the same consistency. At later ages (90 days and greater), with continued moist curing and suitable materials greater strengths may be obtained by use of blended cements. For short curing periods such as are used on ordinary construction, the use of the blended cements resulted in lower compressive strengths.

3. During the period of initial drying, the shrinkage of the blended cement mortars was greater than the shrinkage of the portland cement mortars (two exceptions, see text).

4. The effect of pozzuolanic materials varied with different brands of portland cement.

Readers are referred to the JOURNAL *for April 1931 for discussion which may develop. Such discussion should reach the Secretary by March 1, 1931.*

Tests of Bonding of Floor Finish to Slabs of Haydite and Gravel Concrete

BY F. E. RICHART* AND V. P. JENSEN†

INTRODUCTION

The use of reinforced Haydite concrete for floor slabs has naturally raised the question as to the ability of a finish course of mortar or terrazzo to bond to the slab. Some difficulties have been anticipated by designers due to the tendency for coarse Haydite particles to rise to the surface of the base course and due to the differences in elastic behavior of the topping and base concretes in which the first has a high modulus of elasticity and the second a low one. Questions of this sort have usually been answered through experience with a floor in service and this is without doubt the most satisfactory and convincing answer. The writers know of no laboratory tests that have been made on the bonding of floor finish; such tests could hardly be expected to include differential shrinkages of the two courses over large floor areas, or the effect of hard service, such as heavy trucking, impact, abrasion or vibration. However, in an attempt to devise a test that would show whether there were obvious defects in the bonding, and show them within a short time, the following simple tests were made.

Small reinforced concrete slabs, consisting of a 5-in. base course and ¾-in. finish were tested in flexure on a 4-ft. span with ⅓-point loading. With this arrangement there is produced a relatively high horizontal shear tending to separate base and finish courses in the outer thirds of the span. It was felt that if this type of specimen could be tested to failure without crack-

*Research Associate Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.
†Special Research Assistant, Theoretical and Applied Mechanics, University of Illinois, Urbana Ill.

ing the topping from the base it would indicate that no inherent defects could be attributed to the bonding of the two courses. For comparative purposes, slabs were made using both gravel and Haydite concretes for the base course.

The tests were made at the Materials Testing Laboratory of the University of Illinois as a part of an investigation being carried on in cooperation with the Western Brick Co., Danville, Illinois, manufacturers of "Western" Haydite aggregates. The work is being carried on under the administrative direction of Dean M. S. Ketchum, Director of the Engineering Experiment Station, and Prof. M. L. Enger, Head of the Department of Theoretical and Applied Mechanics, University of Illinois. The Western Brick Co. is represented by an Advisory Committee consisting of I. N. Doughty, Manager of Research, and Frank Payne, Engineer. All of the Haydite aggregates used in the tests were furnished by the Western Brick Co.

OUTLINE OF TESTS

Twelve slabs were made and tested. The design details and dimensions are given in Fig. 1. Two types of concrete were used in the 5-in. slab base; one made with sand and gravel aggregates, the other with fine and coarse Haydite aggregates. Two mixes of each concrete were used, (1) a fairly rich structural mix, 1:2¼:2¾ by volume, and (2) a lean mix, 1:4½:5½, approximating floor fill. Three slabs and three 6 by 12-in. control cylinders were made of each mix. The ¾-in. finish or topping for all slabs was a 1:1:2 mix made with sand and pea gravel using a water-cement ratio of 0.67.

MATERIALS

Universal portland cement passing standard specifications was used. Average tensile strengths of 1:3 standard Ottawa mortar briquets were 290 and 390 lb. per sq. in. at the ages of 7 and 28 days, respectively.

A well graded sand from Lincoln, Ill., having a fineness modulus of 2.70 was used in the gravel concrete bases and in the finish course. Two sizes of Wabash river gravel from Covington, Ind., were used. A pea gravel, with a fineness modulus of 5.98, was used in the finish course and a mixture of this gravel with a

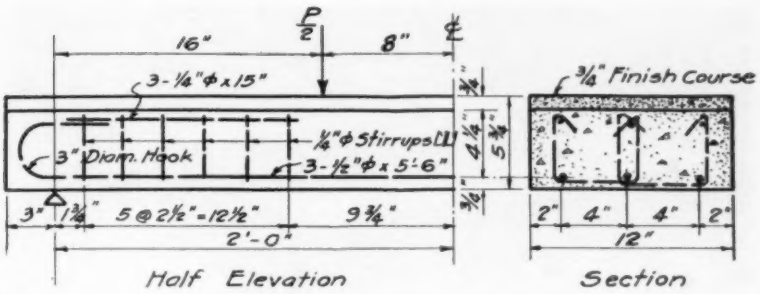


FIG. 1—DETAILS OF TEST SLABS

¾-in. grading, having a fineness modulus of 6.12 was used in the slab concrete.

The Haydite aggregates were the "Western" fine or "A" size, 0—$\frac{3}{16}$-in., with fineness modulus 2.33, and the coarse or "C" size, $\frac{7}{32}$—¾-in., with fineness modulus 5.89.

Each test slab was reinforced with three ½-in. deformed round bars of intermediate grade running the full length of the slab and ending in hooks bent to a 1½-in. radius. These longitudinal bars were tied together in a unit with a double system of ¼-in. round U-stirrups of structural grade spaced 2½-in. apart in the outer thirds of the slab. A typical unit is shown in Fig. 2. None of the reinforcement projected into the finish course of the slab.

MAKING OF SLABS

The slabs were poured in bottomless wooden forms on a sheet of building paper resting on the laboratory floor. Slight leakage

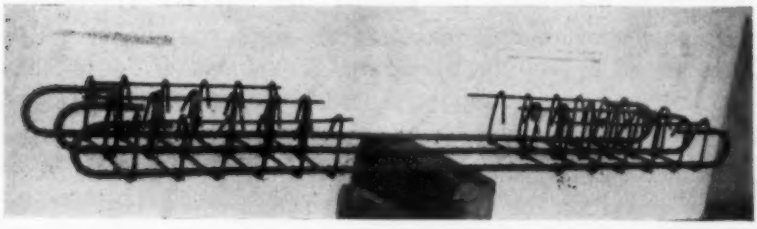


FIG. 2—TYPICAL REINFORCEMENT UNIT

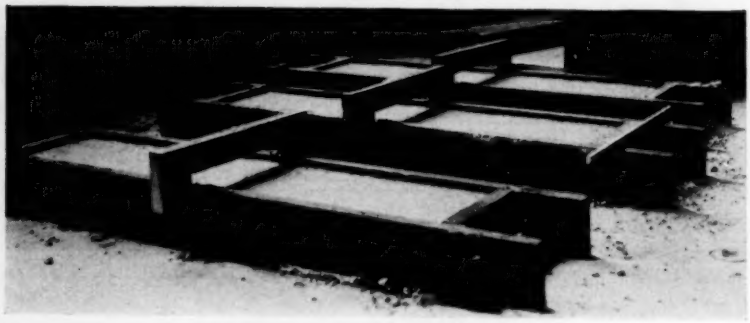


FIG. 3—BASE SLABS

of water and mortar occurred at the bottom edges, particularly with the 1:10 concretes despite the fact that these concretes gave low slumps. The base concrete was mixed in a one-bag batch mixer, hauled to the form in wheel-barrows, shovelled into place and spaded uniformly around the reinforcing steel. The concrete was struck off to a level ¾-in. below the edges of the form. Some settlement occurred after this screeding, especially in the 1:10 mixtures. The settlement was more noticeable in the 1:10 Haydite than in the gravel concretes. After the concrete had stiffened, the surface was struck with a wood float and left exposed to the air until the following day. Three forms are shown in Fig. 3, with the base slabs struck off ready for the finish course.

The finish course was made and placed in general accordance with the recommended practice given in the Report[1] of Committee 802 on Concrete Floor Finish, of the American Concrete Institute. The chief departure from the recommended practice was in the use of a fine sand which was permitted because it was felt that the primary reason for using a coarse sand for floor finish is to improve the wearing quality of the surface, a quality not to be measured in these tests, and that any change in the physical characteristics of the topping, caused by the use of a fine instead of a coarse sand, would have little effect upon the test results.

Twenty-four hours after the base slab was poured, the surface was prepared for the finish course. This was done by wetting

[1] "Good Practice in Concrete Floor Finish" John G. Ahlers, JOURNAL of the Am. Con. Inst., March. 1930, *Proceedings*, Vol. 26, p. 520.

the surface thoroughly and then brushing on a thin coat of neat cement mortar with a wire brush, care being taken to mop up any pool of surface water before brushing in the mortar. The finish was placed immediately after the application of the mortar coat. It was tamped uniformly over the surface of the slab and into the corners of the form to insure a full surface of adhesion in every slab. The surface was then struck off level with the edges of the forms. and compacted and smoothed with a wood float. The surface was trowelled with a steel trowel one to two hours after being struck off.

The average measured thickness of finish used with the four concretes was as follows: 1:5 gravel, 0.74 in.; 1:10 gravel, 0.75 in.; 1:5 Haydite, 0.75 in.; 1:10 Haydite, 0.90 in.

CURING

The forms were stripped one day after the finish course was placed, and the slabs were stored under wet burlap for about five days; they were then removed to the moist room for standard moist curing at 70° F. All cylinder molds were stripped after 48 hours and the cylinders cured in the moist room until the time of test, when they were tested moist.

TESTING OF SLABS

The slabs were removed from the moist room when 28 days old and tested within 6 hours after removal. Loose moisture was wiped from the surfaces and a coating of whitewash was applied to the sides and ends to aid in the detection of cracks. A very marked feature of the 1:10 Haydite concrete slabs was that the moisture present in the base of the slab kept coming to the sides so rapidly that the whitewash was kept from drying during the day of the test.

A general view of the testing apparatus is shown in Fig. 4, which includes a typical slab after test. The work was done in an Olsen testing machine of 300,000-lb. capacity. The load was applied through a spherical bearing block to a grillage made up of four 4-inch channels and thence through rollers and bearing plates to the third-points of the span. The supports permitted free rotation at both ends and horizontal movement at one end. All bearing plates were set in plaster of paris. Load was applied

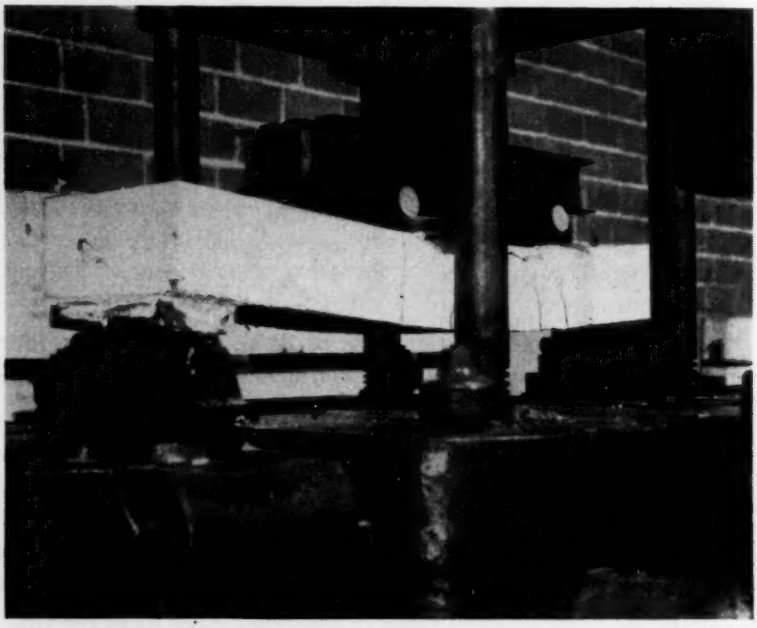


FIG. 4—TESTING APPARATUS SHOWING TYPICAL SLAB AFTER TEST

in increments of 500 to 1000 lb. at the slowest testing speed of the machine. A close watch was kept for cracking of the slab and any indication of loosening of the topping from the base, but no strain measurements were taken. After the steel had passed its yield point, as indicated by a dropping of the weighing beam and opening of tension cracks, a faster test speed was maintained until final rupture occurred.

The modulus of elasticity of the concrete was determined from stress-strain measurements on the control cylinders.

PHENOMENA OF TESTS

With one exception the slabs failed initially by tension in the longitudinal steel. The single exception was a Haydite slab of 1:10 mix which failed initially by diagonal tension at a load of 13,900 lb. In this slab the yield point of the longitudinal steel was reached at a load of 15,700 lb. and the ultimate load carried was 16,300 lb. At failure, a piece split off of the side of the slab

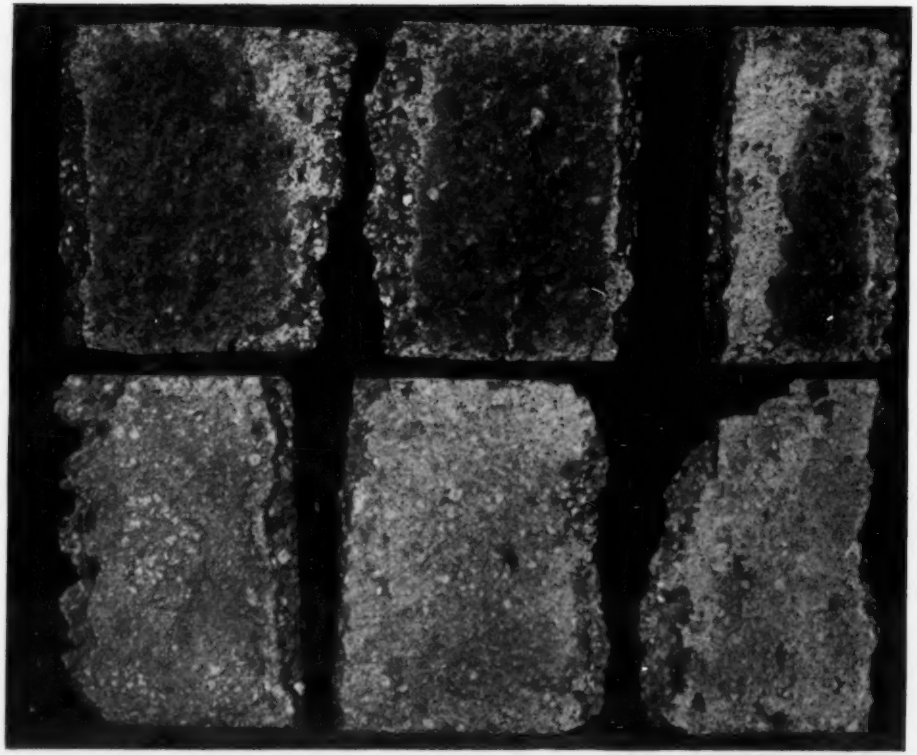


FIG. 5—BOTTOM VIEW OF PIECES OF FINISH WHICH CRACKED OFF AT FINAL FAILURE

(Top three from slabs of Haydite base; bottom three from slabs of Gravel base.)

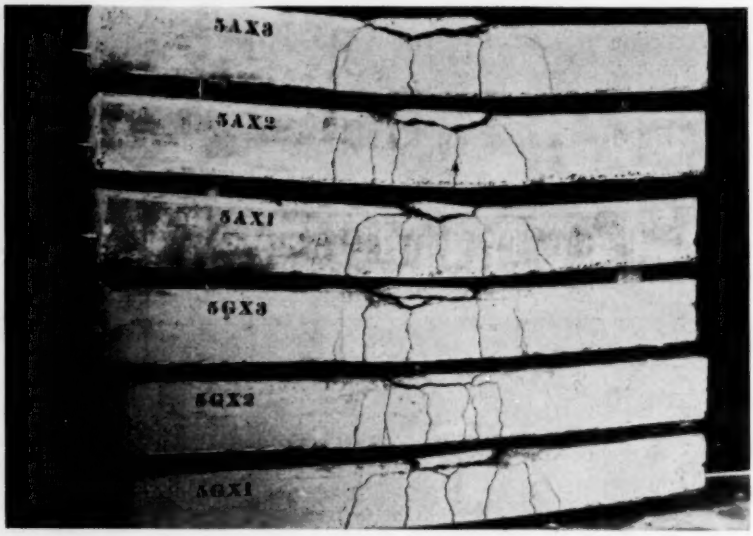


FIG. 6—ALL SLABS OF 1:5 MIX AFTER TEST
(Top three slabs of Haydite base; bottom three slabs of gravel base.)

(No. 10AX3, Fig. 7) near the end, and it was evident that a large amount of slipping of bar and compression of the concrete inside the hook had occurred. There was also evidence of large distortion of the weak and compressible concrete due to high bearing pressure fro n the cast iron plate at the support. While the large amount of movement of the hooked bar indicates that the diagonal cracking may have been due to slip of bar, splitting at the end of the slab occurred well after the yield point of the longitudinal steel had been reached. Complete test data are given for all slabs in Table 1. In no tests did the finish crack loose from the base before the ultimate load was reached. The finish in both Haydite and gravel concrete slabs remained integral with the base with deflections considerably in excess of one inch on a 4-ft. span. At final failure, with deflections of about 1½ inches, crushing of the ¾-in. finish course occurred in the middle third near one of the load points and a section of the topping split off. The sections which split off from the Haydite slabs had portions of the base adhering to the topping, the frac-

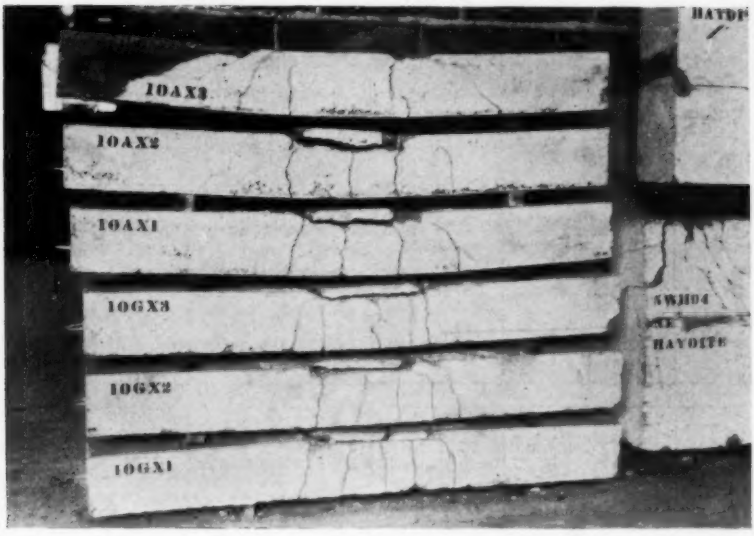


FIG. 7—ALL SLABS OF 1:10 MIX AFTER TEST
(Top three slabs of Haydite base; bottom three slabs of gravel base.)

ture being mostly below the topping. Where the base was of gravel concrete the fracture occurred at the bonding surface between the base and topping. Fig. 5 shows a bottom view of the pieces of topping which split off from the 1:5 concrete slabs. A general view of all the 1:5 slabs is shown in Fig. 6 and of all the 1:10 slabs in Fig. 7. In each figure the upper three slabs are of Haydite concrete and the lower three of gravel concrete.

DISCUSSION OF TEST RESULTS

It is quite evident from these tests that there was no great difference in the bonding of the floor finish to the Haydite and gravel concrete bases. If anything the final crushing failures showed a more definite plane of cleavage between the finish and the gravel concrete than between finish and the Haydite concrete. With care taken in applying the floor finish it is evident that very satisfactory bonding may be secured with either type of base concrete, since in no case was there any separation between finish and base in the outer thirds of the span even at failure of

TABLE I—DATA AND RESULTS OF SLAB TESTS.

Mix by Volume	W/C by Vol.	Slump In.	Flow %	Unit Wt. Fresh Concrete Lb. per Cu. Ft.	Slab Tests			6" x 12" Cyl.	
					Slab No.	Load at Yield Lb.	Ultimate Load Lb.	Compr. Strength Lb. per Sq. In.	Initial Mod. of Elast. Thousands of Lb. per Sq. In.
Base Concrete Made with Natural Sand and Gravel									
1:2¼:2¾	0.90	5.8	220	146.8	5GX1	15,700	19,700	4390	3720
					5GX2	15,800	18,900		
					5GX3	16,200	18,700		
					Average	15,900	19,100		
1:4½:5½	1.70	1.5	213	142.9	10GX1	15,700	16,700	1260	3220
					10GX2	15 800	17,900		
					10GX3	16,200	18,100		
					Average	15,900	17,600		
Base Concrete Made with Fine and Coarse Haydite									
1:2¼:2¾	1.02	5.3	235	97.1	5AX1	16,000	19,200	3260	1860
					5AX2	16,400	20,300		
					5AX3	15,900	20,100		
					Average	16,100	19,900		
1:4½:5½	2.02	1.9		90.5	10AX1	15,700	18,300	620	920
					10AX2	15,500	17,400		
					10AX3*	*	16,300		
					Average	15,600	17,300		
Finish Course Made with Sand and Pea Gravel									
1: 1 : 2	0.66	5.0		149.2				5540	4030

All slabs except 10AX3 failed initially by tension in the longitudinal steel.
*Slab 10AX3 failed initially by diagonal tension.

the slab. The separation at the point of final crushing of the top surface of the slab might be expected in view of the great distortions and deflections existing in the slab at failure.

An unexpected feature of the tests was found in the fact that the slabs of lean concrete carried practically as much load as those of the rich mixture. It is evident that the strong ¾-in. finish course developed most of the compressive flexural strength of the slabs made with the lean concrete. With roughly 1 per cent of reinforcement in all slabs it appears that the compressive concrete stress was not excessive until the yield point of the reinforcement has been exceeded; it is of interest that a margin of safety existed in the rich layer of the finish course.

It may be concluded that no weakness in bonding between the floor finish and either of the base concretes was indicated by the tests. On the contrary, the tests give support to the practice

of including the floor finish in computing the effective depth of the slab for design purposes. Although the tests indicate that the finish course produced much more slab strength than an equal thickness of base concrete, the writers do not feel that the amount of data secured justify any new method of analysis or design beyond the inclusion of the finish thickness in computing effective depth of slab. So much depended upon the quality of the finish course in the slabs made with lean concrete that any defect in the finish would have greatly decreased the slab strengths. Furthermore, the 1:10 concretes offered such low resistance to local compression that it is surprising that sufficient anchorage was effected to develop the elastic strength of the longitudinal steel.

As noted in the introduction, the tests described give no indication of the effect of shrinkage or of heavy service conditions on a large floor slab; they are quite reassuring in that no obvious defects in bonding of finish to base course was in any way indicated. In the absence of actual service tests of long duration, these experiments indicate that bonding of a floor finish is equally feasible with base slabs of gravel and Haydite concrete, providing careful workmanship is employed.

Readers are referred to the JOURNAL *for April, 1931, for discussion which may develop. Such discussion should reach the Secretary by March 1, 1931.*

Deflection of Reinforced Concrete Members

*Progress Report of Committee 307**

BY T. D. MYLREA,† AUTHOR-CHAIRMAN

PURPOSE

The scope of this report will be limited for the present to a study of the beam and slab type of construction, leaving the flat slab and other forms of construction for future investigation.

An expression for the calculated deflection of a reinforced concrete member may be desirable for one or more of the following purposes: (1) To provide a criterion of acceptability for a given structure, (2) for the computation of the camber required during construction, (3) as a basis for the development of methods of solving statically indeterminate structures. Obviously, to be satisfactory for any of these purposes, an expression should be one from which deflections may be predicted from computed stresses, that is, one which is independent of measured deformations. It is quite possible that such an expression might fulfil one purpose satisfactorily without necessarily being of much use in the others, and for this reason all formulas were studied and data digested with a view to their applicability to any of the above uses. It is hoped that some of the conclusions may be of assistance to the sub-committee of Committee 501 dealing with load tests.

FORMULAS

Two general types of formulas are commonly known—those of Turneaure[1] and those of Maney. Turneaure's formulas are not derived along conventional lines, and hence have not yet

†Consulting Engineer, Pittsburgh, Pa.

*This report was on the program of the 26th Annual Convention but in the absence of its author was not presented. It has since been approved by P. J. Freeman and A. H. Saurbrey, as critic members of Committee 307.

[1] "Principles of Reinforced Concrete Construction," by Turneaure and Maurer, 3rd Ed. Chap. VI.

been subjected to a critical study. Maney's formula, as originally propounded,[2] is in the following form:

$$\Delta = C\frac{L^2}{d}(e_s + e_c) \quad \ldots\ldots (1)$$

In this formula C is a constant depending upon the method of loading and end restraint, and the rest is a matter of plane geometry. Following the usual procedure in expressing nominal concrete and steel stresses, its author rewrote it[3] as follows:

$$\Delta = C\frac{L^2}{dE_s}(f_s + nf_c) \quad \ldots\ldots (2)$$

Since to be of service in predicting deflections a formula based on computed stresses is desirable, Formula 2 is in a much more convenient form than Formula 1.

Prof. Hardy Cross and C. A. P. Turner have proposed a modification of Formula 2, which may be expressed:

$$\Delta = C\frac{L^2 f_s}{d(1-k)E_s} \quad \ldots\ldots (3)$$

However, since k must be computed from the relation between f_s and nf_c there seems to be no special advantage in this form.

Formula 2, therefore, was adopted in making all comparisons.

ASSUMPTIONS AND THEIR EFFECTS

Computed deflections are proportional to the quantity within the parenthesis in Formula 2, and the assumptions used in the flexure formulas will of course affect the value of the terms in this quantity. The value of f_s is based upon the assumption that concrete can take no tension. Yet, at the beginning, only the steel crossing a crack can have its computed value. Later, as bond breaks down, stresses in the steel will be high where computations, indicate the existence of low stress. This condition, by its nature, seems to be closely related to the sum of the crack widths, from which deductions may be made later. End slip of bar will add to crack widths.

The value of n, depending upon a value of E_c which is assumed to be constant from the neutral axis to the extreme compression fiber, will have a much greater effect upon f_c than upon f_s. Consulting any table giving values of j for various values of n will

[2]G. A. Maney, "Relation between deformation and deflection in reinforced concrete beams." Proc. Am. Soc. Test. Materials, 1914, p. 310.
[3]Parcel and Maney, "Statically Indeterminate Structures," p. 48-51, Wiley.

show than for a change of 25 per cent in the values of n the corresponding change in j, and consequently in f_s, is generally less than 2 per cent. And this condition does not change much even when E_c is assumed to be variable, as in the so-called "parabolic formulas. Changes in f_c are inversely proportional to changes in n.

Another factor which materially affects the computed deflection of actual structures, as distinguished from isolated specimens, is the assumed width of T. This has a direct bearing on the magnitude of f_c; and until correct T values can be assigned it is futile to conduct load deflection tests with the object of passing upon the merits of a completed piece of construction. This matter will be given further consideration when available test data have been discussed.

Effect of n upon Δ.

Since Δ varies directly as $(f_s + nf_c)$ it is easy to study the part that n plays in computed deflections. As n increases f_c decreases, and vice versa, so that the variation in the term nf_c is much less than the variation in n. For example, assume a beam in which $f_s = 18{,}000$, $f_c = 800$ and $n = 15$. In this beam $p = 0.0089$ and

$$f_s + nf_c = 18{,}000 + 15 \times 800 = 30{,}000.$$

Now had the value of n been 12, in a beam with this steel ratio, f_s would have been 17,800 and f_c would have been 870.

Here $f_s + nf_c = 17{,}800 + 12 \times 870 = 27{,}840.$

In other words, with a change of 25 per cent in the value of n there is a change in deflection of only 7.75 per cent.

The discrepancy is more noticeable in the case which is more usual in building construction, i. e., where the concrete stresses are low because of wide T-flanges. Consider, for example, a beam in which $f_s = 20{,}000$, $f_c = 500$, and $n = 15$. In this beam $p = 0.0034$ and

$$f_s + nf_c = 20{,}000 + 15 \times 500 = 27{,}500.$$

If the value of n had been 12, then f_s would have been 19,850 and f_c would have been 545. Here

$$f_s + nf_c = 19{,}850 + 12 \times 545 = 26{,}400,$$

and in this case a difference of 25 per cent in the value of n would have occasioned only 4.17 per cent difference in deflection.

In flat slab construction where p is always low, the difference in deflection might be even less.

Now in buildings under test the deflections are of the order of hundredths of an inch. An investigator would be hardy indeed to condemn a building as having concrete 25 per cent below par when his deflections varied only a few per cent from some predetermined few hundredths of an inch. The author advances this point for the consideration of the sub-committee on load tests.

On the other hand, the very fact that deflections are not proportionately influenced by variations in concrete strength is of much value in computing camber and in sustaining confidence in computations involving continuity. Accidental variations in strength will not necessarily vitiate computations.

GENERAL OBSERVATIONS ON AVAILABLE DATA

So far the following literature has been examined: University of Illinois Engineering Experiment Station Bulletins No. 4, 12, 28, 29, 64 and 71; United States Bureau of Standards Technological Paper No. 314, and Deutscher Ausschuss für Eisenbeton Heft No. 38. In addition, the author has at hand the data from the tests of 10 large T-beams. These papers with the exception of Bulletin 314, all deal with isolated simply supported rectangular or T-beams. In Bulletin 71 and Heft 38, end slip of the main reinforcing is recorded, the significance of such slip having by this time become evident. In Bulletin 314 the effects of slip are noted carefully. In some of the beams tested by the author slip was permitted and in others it was prevented. A comparison of deflections is being made.

A general characteristic of nearly all tests is that loads were applied progressively to failure, stopping at various increments just long enough to make the required observations and measurements. University of Illinois Bulletin No. 4 has some valuable curves showing the effect of repeated applications of the same load as well as curves showing what took place when each load following a release was higher than the one previous.

COMPARISON OF ACTUAL AND COMPUTED DEFLECTIONS

Published load deflection curves made from actual test data where loads were carried progressively to failure are all curved. For quick reference, the reader is referred to a typical curve of

this nature on page 281 of the January issue of the JOURNAL of the American Concrete Institute. It will be noted that at low loads, say below 2500 lbs. the curvature is sharp, while the remainder of the curve might almost be represented by a straight line. About the middle of the curve is a point labelled "first crack." This test was made before it was discovered at the University of Wisconsin that actual cracks occurred long before they were visible to the naked eye. On the basis of the reported tensile strength of the concrete used, the first crack probably occurred at a load of between 1000 and 1500 lbs.—i. e. approximately at the beginning of the sharp curvature; and it would appear that the shape of the curve below the 3000 lb. load is largely due to the formation and development of fine cracks. The beam is reported to have failed in tension. Had deflection readings been continued beyond the loads shown, a comparison of similar curves indicates that this curve would have deviated again from the nearly straight line condition bending over to the horizontal and perhaps below it. Such a deviation also accompanies end slip of bar as may be seen in the graphs on pages 229-238 of Bulletin No. 71. It is of interest to note in the tests reported in Bulletin No. 71 that while slip of bar along the middle portions of the beams occurred at very low loads, end slip did not make its appearance until steel stresses approximately as high as those in use for general design purposes had been attained. There is a portion of the curve, then, which may be of special significance.

Turning again to Formula (2), since f_s and nf_c, the computed fibre stresses, are directly proportional to the applied load, the curve of computed deflections should be a sloping straight line passing through the origin. Computations in many of the typical cases indicate that this line intersects the curve of actual deflections at such a point as would give a computed steel stress of from 18,000 to 28,000 lb. per sq. in.—generally at about the point marking first end slip. Where end slip is prevented the higher values obtain.

It would be easy to write an equation representing such a curve as the one shown on page 281 of the JOURNAL of the A. C. I., January, 1930, but an equation of this sort would serve no useful purpose, since no portion of this curve would ever be duplicated in this beam. Suppose, for example, that in testing for the first

time the load had been progressively applied up to 4,000 lb. The load-deflection diagram would be as shown in the figure up to point "A." Upon the release of the load the beam would recover, and if the release were gradual an "unload" deflection diagram would be a straight line from point "A" back to the point of zero load but not to the point of zero deflection. There would be a permanent deflection of about .01 in. as indicated by the "set curve." Experiments by the author, in which loads were repeated as many as 55 times, indicate that if this 4000 lb. load were repeatedly applied and released, the load and unload curves would follow up and down this sloping line. Similarly, for any other load, for instance, the 7000 lb. load, the load and "unload" curve would follow up and down an approximately straight line from about the point marked "first crack" to a point on the deflection axis of about .025 in. The process apparently would be repeated until the magnitude of the load was such that end slip of the bar or progressive permanent deformation of the concrete took place. This condition would be apparent by the fact that repetition of the load would produce a "saw-tooth" curve with the peaks all at the same altitude. If any load were applied once and then released, the "unload" curve would be approximately a straight line, but upon the next application of a higher load, the upper end of the ordinate representing the resulting deflection would lie on the curve given on page 281. It is as yet an open question just to what extent tha permanent deformation of the concrete affects the deflection, and this, of course, leads at once to the study of long-continued loads and plasticity. It is apparent, too, that if a beam has once been subjected to a high load, the load-deflection curve for any subsequent progressively applied load will lie below the typical load-deflection curve and will probably more nearly coincide with the curve of computed deflections regardless of the assumed value of n. If it is possible that a rational equation can be found representing the points on a deflection curve plotted for loads following the application of a high load, it may also be possible, (since the effect of the value of n is not of major importance, and since the value of E_c for concrete stresses usually encountered in design does not vary much from the neutral axis to the extreme compression fibre of a beam), that by the test of a bay in a building

the width of floor slab performing flange duty for a beam may be found deductively from Formula (2). For example, if such a test could now be applied to the George Mason Hotel (see Prof. Slater's article in the January 1930 issue of the JOURNAL of the A. C. I.) with loads below 238 lb. per sq. ft., this building, because of the extreme flange width obtainable, might well serve such a purpose.

BEAMS OF UNUSUAL SHAPE

The above observations are all based upon the tests of rectangular or T-beams. The results reported in Technological paper No. 314 however, seem to indicate that any conclusions drawn above may not apply to beams differing much in shape from the usual rectangular or T-shaped beam. In the beams tested for the Shipping Board, from the very beginning in most cases, the actual deflections exceeded the computed deflections—just the reverse of what has been previously discussed. This held true even when actual measured deformations were substituted in Formula (1), and hence when the agreement ordinarily would have been close throughout the whole range of the test. The authors ascribe this discrepancy between the actual and computed deflections to shearing deflection and vertical opening between cracks. From this it seems possible that the neglect of shearing distortions may have a relatively greater effect in deep concrete beams than in short deep beams of other structural materials, and that such deflections may not mean an unsafe structure. It means, too, that any deductions based on tests of such beams in a completed structure must be made with extreme caution.

CONCLUSION

In presenting this progress report the author has attempted to cover the work already done, and to outline the plan for future work, and will appreciate constructive criticism or suggestions from any interested members.

Readers are referred to the JOURNAL *for April 1931 for discussion which may develop. Such discussion should reach the Secretary, by March 1, 1931.*

Continuous Beams and Frames in Building Construction

BY T. U. BERG*

The object of this paper is to present a ready means of determining the bending moments and shearing forces for continuous beams and frames of approximately equal spans. Except as indicated in Appendixes 1 and 2, this is not a general method of solving rigid frames, but a presentation of simple formulas and constants from which the designer of reinforced concrete can directly read maximum bending moments with due consideration for the influence of dead as well as live loads.

The negative moments at supports for critical loadings can easily be determined from the accompanying table or graphs. (The graphs have all been prepared from values given in Table 3). The problem is then reduced to a statistically determinate one, and the moment curves are drawn in the usual way by combining the negative support moments with the moments due to the loading acting on the simply supported span.

The moment at any support and for any system of loading can be written: $M = C_D \cdot K_D + C_S \cdot K_S$ where K_D and K_S are constants respectively for dead and live loads as given in Table 1, Fig. 2. It will be noted that these constants are similar to what are generally called "moment area constants." They are: $1\frac{1}{2}$ times the area of the moment curve for a simple supported beam divided by the span. For combinations of loadings, determine one value for each type of loading and add them together.

C_D is a coefficient to be read directly from the table (or graph). For convenience, D will be used in the following instead of C_D.

*Structural Engineer, New York City.

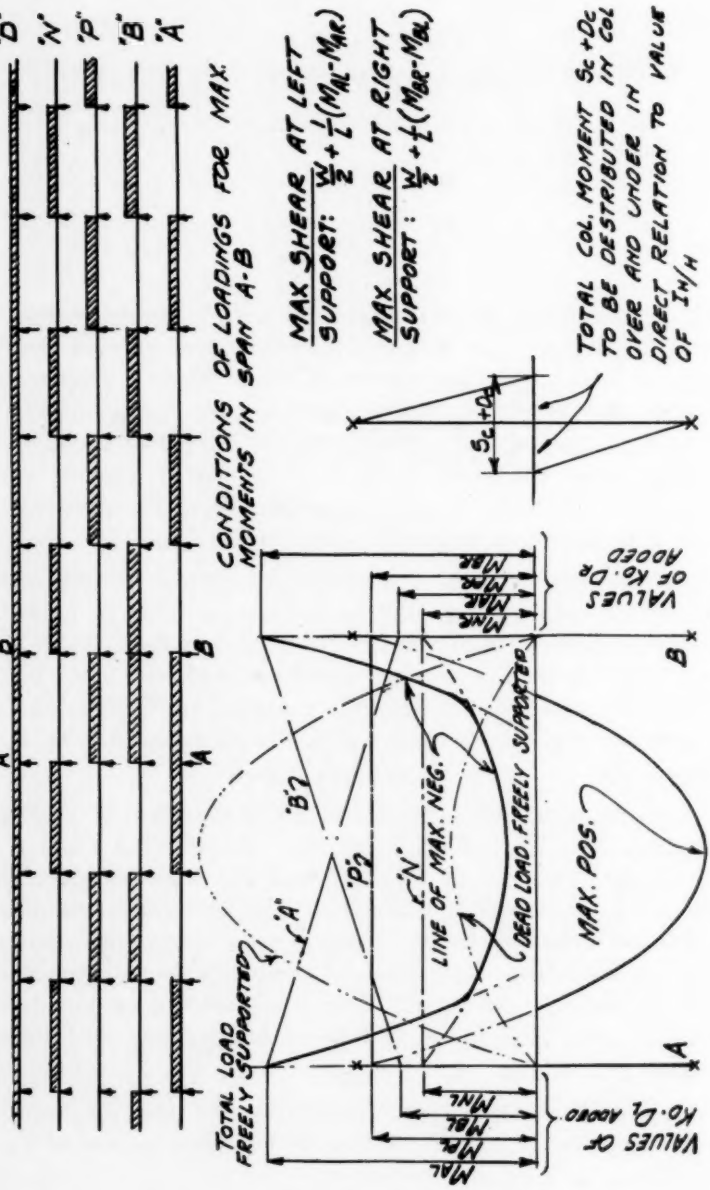

"D"
"N"
"P"
"B"
"A"
A
B
CONDITIONS OF LOADINGS FOR MAX. MOMENTS IN SPAN A-B
MAX SHEAR AT LEFT SUPPORT: W/2 + 1/L (MAL - MAR)
MAX SHEAR AT RIGHT SUPPORT: W/2 + 1/L (MBR - MBL)
TOTAL COL. MOMENT Sc + Dc TO BE DESTRIBUTED IN COL OVER AND UNDER IN DIRECT RELATION TO VALUE OF I/H
Sc + Dc
TOTAL LOAD FREELY SUPPORTED
"A"
"B"
"P"
"N"
LINE OF MAX. NEG.
DEAD LOAD. FREELY SUPPORTED
MAX. POS.
VALUES OF Ko. DL ADDED
VALUES OF Ko. DR ADDED
MAL
MAR
MBL
MBR
MNL
MNR

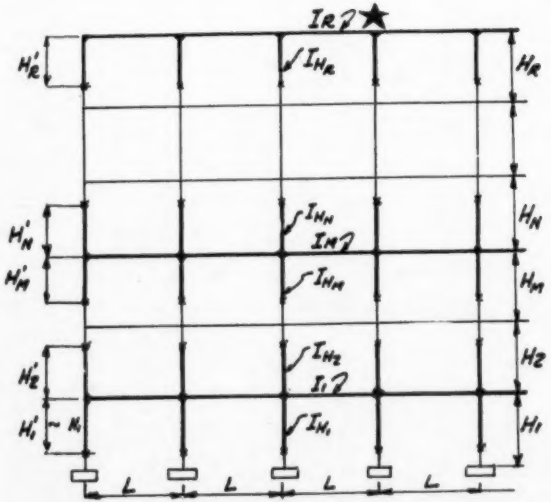
FIG. 1

TABLE 1

VALUES OF K_0 & K_3	
LOADING	K
MOMENT AREA F, SPAN L, ANY SYM. LOAD	$1.5\frac{F}{L}$
W, $\frac{1}{8}WL^2$	$\frac{1}{8}WL^2$
P, $\frac{1}{4}PL$	$\frac{3}{16}PL$
P P, $\frac{1}{3}PL$	$\frac{1}{3}PL$
P P P, $\frac{1}{8}PL$, $\frac{1}{2}PL$	$\frac{15}{32}PL$
P P P P, $\frac{2}{5}PL$, $\frac{1}{5}PL$	$\frac{3}{5}PL$
W, $\frac{1}{6}WL$	$\frac{5}{32}WL$

TABLE 2

$\frac{I_N}{H'}\frac{L}{I}$	HORIZ. SCALE	$\frac{H'}{H}$
0	0	
.1	1.30	0.76
.2	2.40	"
.3	3.20	"
.4	3.90	0.77
.5	4.50	"
.6	5.10	0.78
.7	5.50	"
.8	5.90	"
.9	6.30	0.79
1.0	6.55	"
1.5	7.70	0.80
2.0	8.50	0.81
3.0	9.50	0.82
4.0	10.00	0.83
6.0	10.60	0.84
8.0	11.00	"
10.0	11.20	0.85
∞	12.20	



FOR DIFFERENT VALUES OF $\frac{I_N}{H'}$ FOR COLUMNS BELOW AND ABOVE FLOOR, USE AVERAGE VALUE . $\frac{1}{2}\left(\frac{I_{NM}}{H'_M}+\frac{I_{NN}}{H'_N}\right)$

★ FOR COLS. BELOW FLOOR ONLY USE HALF VALUE ONLY. $I_N/H' = 0.5 \cdot I_{NR}/H'_R$

FIG. 2

C_S is a coefficient depending on the arrangement of live loads, i. e. which spans are loaded or unloaded.

As our aim is to determine maximum values for moments, we must consider the following cases:

Fixed Loads. C_S for live loads of similar nature to dead loads, such as permanent brick walls etc., should be taken the same as for dead loads. $C_S = C_D = D$.

Maximum Positive Moments. This condition occurs for the system of loading as shown in Fig. 1, mark P, i. e. spans alternately loaded and unloaded. For this case we will substitute the mark P for live load coefficient instead of C_S, and the support moment can be written: $M_P = D \cdot K_D + P \cdot K_S$.

Maximum Negative in Middle of Span. See loading as shown in Fig. 1, mark N. Note that the span investigated is unloaded (dead or fixed loads only). All other spans are alternately loaded and unloaded. Use N as mark for coefficient instead of C_S, and we have for support moments: $M_N = D \cdot K_D + N \cdot K_S$.

Maximum Negative Near Support. For a span A-B, the maximum moment at support B will occur for loading same as shown in Fig. 1, mark B, i. e., spans at both sides of support B loaded, other spans alternately unloaded and loaded. Substitute B for coefficient C_S and we have: $M_B = D \cdot K_D + B \cdot K_S$.
In a similar manner we have for support A, loading A, Fig. 1: $M_A = D \cdot K_D + A \cdot K_S$.

In order to draw an accurate curve for moments near A, and B, it is necessary also to determine the moment at the opposite end. Thus, for maximum at B, a moment should also be figured for the other end A. To make this clear, the constants have been given the following marks:

Loading A (maximum moment at support A):

C_S (at left support A) $= A_L$, $M_{AL} = D \cdot K_D + A_L \cdot K_S$

C_S (at right support) $= A_R$, $M_{AR} = D \cdot K_D + A_R \cdot K_S$

Loading B (maximum moment at support B):

C_S (at left support A) $= B_L$, $M_{BL} = D \cdot K_D + B_L \cdot K_S$

C_S (at right support) $= B_R$, $M_{BR} = D \cdot K_D + B_R \cdot K_S$

It should be noted that for each support, values must be determined for M_P, M_N, M_A and M_B, but they are all simple slide rule operations.

K=I/A=I	LOADING		MARK	2-SPANS		3-SPANS				4-SPANS				5-SPANS						TYPICAL		
				I		I		II		I		II		I		II		III				
				L	R	L	R	L	R	L	R	L	R	L	R	L	R	L	R	L	R	
0	DEAD LOADS		D	0	1	0	.400	.100	.400	0	.157	.157	.571	0	.142	.842	.632	.632	.632	.667	.667	K_D
	ALT. SPANS LOADED		P.N	0	.500	0	.400	.400	.400	0	.429	.429	.286	0	.421	.421	.316	.316	.316	.333	.333	
	MAX. AT SUPPORTS	LEFT	A	—	—	—	—	.933	.278	—	—	.964	.143	—	—	.957	.172	.890	.163	.909	.181	K_S
		RIGHT	B	0	1	0	.933	.278	.933	0	.964	.286	.857	0	.957	.278	.890	.163	.890	.181	.909	
0.10	DEAD LOADS	BEAMS	D	.087	.957	.100	.798	.768	.768	.097	.839	.817	.592	.098	.829	.804	.634	.641	.641	.667	.667	K_D
		COLUMNS	D_c	.087	0	.100	-0.030	—		.097	-0.022			.098	-0.025		0.007	—		0		
	POSITIVE DISTR.		P	.123	.520	.130	.434	.434	.434	.129	.453	.460	.339	.129	.447	.455	.358	.366	.366	.377	.377	
	NEGATIVE DISTR.		N	+.036	.436	+.030	.364	.334	.334	+.031	.386	.356	.253	+.032	.382	.349	.276	.275	.275	.290	.290	
	MAX. AT SUPPORTS	LEFT	A	.123	.520	.130	.434	.196	.306	.129	.453	.273	.205	.129	.447	.918	.224	.167	.222	.884	.230	K_S
		RIGHT	B	.087	.957	.091	.905	.306	.896	.089	.929	.322	.839	.090	.924	.315	.162	.222	.867	.230	.884	
	LIVE LOADS on COLS		S_c	.123	.084	.130	-0.102	—		.129	-0.096	.086		.129	-0.098		0.090	—		0.087		
0.40	DEAD LOADS	BEAMS	D	.249	.877	.270	.775	.721	.721	.266	.804	.753	.623	.271	.800	.747	.643	.656	.656	.667	.667	K_D
		COLUMNS	D_c	.249	0	.270	-0.054	—		.266	-0.051			.271	-0.053		0.013			0		
	POSITIVE DISTR.		P	.321	.554	.334	.495	.495	.495	.333	.502	.516	.435	.338	.500	.514	.442	.455	.455	.468	.468	
	NEGATIVE DISTR.		N	+.072	.323	+.064	.290	.225	.225	+.067	.302	.237	.189	+.067	.300	.233	.201	.201	.201	.208	.208	
	MAX. AT SUPPORTS	LEFT	A	.321	.554	.334	.495	.132	.315	.333	.502	.153	.323	.337	.500	.150	.330	.824	.333	.333	.333	K_S
		RIGHT	B	.249	.877	.256	.150	.315	.832	.253	.162	.395	.803	.253	.160	.391	.825	.333	.824	.333	.833	
	LIVE LOADS on COLS		S_c	.321	.230	.334	-0.270	—		.333	-0.265	.246		.338	-0.267		0.241			0.250		
1	DEAD LOADS	BEAMS	D	.400	.800	.414	.759	.690	.690	.412	.765	.706	.647	.412	.764	.703	.653	.663	.663	.667	.667	K_D
		COLUMNS	D_c	.400	0	.414	-.069	—		.412	-0.059	—		.412	-0.061		0.010	—		0		
	POSITIVE DISTR.		P	.473	.582	.483	.552	.552	.552	.482	.553	.563	.521	.482	.553	.563	.523	.533	.533	.533	.533	
	NEGATIVE DISTR.		N	+.073	.218	+.069	.207	.138	.138	+.070	.211	.142	.126	+.067	.211	.141	.131	.131	.131	.133	.133	
	MAX. AT SUPPORTS	LEFT	A	.473	.582	.483	.552	.775	.467	.482	.553	.786	.436	.482	.553	.785	.437	.776	.439	.781	.439	K_S
		RIGHT	B	.400	.800	.404	.790	.467	.775	.402	.795	.471	.765	.402	.795	.469	.769	.439	.776	.439	.781	
	LIVE LOADS on COLS		S_c	.473	.363	.483	-0.413			.482	-0.411	.395		.482	-0.412		0.402			0.400		
10	DEAD LOADS	BEAMS	D	.625	.687	.626	.686	.667	.667	.626	.686	.667	.666	.626	.686	.667	.666	.667	.667	.667	.667	K_D
		COLUMNS	D_c	.625	0	.626	-0.019			.626	-0.019	—		.626	-0.019		0			0		
	POSITIVE DISTR.		P	.644	.647	.645	.646	.646	.646	.645	.646	.647	.645	.644	.646	.646	.645	.646	.646	.646	.646	
	NEGATIVE DISTR.		N	+.019	.040	+.019	.040	.020	.020	+.019	.040	.020	.020	+.019	.040	.021	.021	.021	.021	.021	.021	
	MAX. AT SUPPORTS	LEFT	A	.644	.647	.645	.646	.686	.627	.645	.646	.687	.626	.645	.646	.687	.626	.687	.626	.687	.626	K_S
		RIGHT	B	.625	.687	.625	.687	.627	.686	.625	.687	.627	.636	.625	.687	.627	.686	.626	.687	.626	.687	
	LIVE LOADS on COLS		S_c	.644	.608	.645	-0.625			.645	-0.625	.624		.645	-0.625		0.625			0.625		

TABLE 3

K_D and K_S are assumed to be of constant value for all spans, and $D \cdot K_D$ is figured only once for each support. See combined moment curves, Fig. 1. It is convenient to draw one diagram for total load on a simply supported beam to be used for P, A, and B lines, and one diagram for dead loads on a simple supported beam to be used for N lines. With the aid of dividers all moment curves can be drawn in a short time.

Shearing Forces. The maximum shear at any support equals the shear for a simply supported beam plus a value depending on the slope of the A and B *lines.* This addition for left support is $1/L \cdot (M_{AL} - M_{AR})$, and for right support, $1/L \cdot (M_{BR} - M_{BL})$.

Column Moments. Maximum moments for columns are given in Table 3 or in the graphs in the following manner:

Coefficient for dead load, D_C
Coefficient for live load, S_C
Column moment, $M_C = D_C \cdot K_D + S_C \cdot K_S$

Moments have been considered positive if they turn in a clockwise direction.

The Basic Frame. The coefficients D, P, N, A and B are dependent upon the form of the frame and stiffness of beams and columns. The main table No. 3 has been figured for frames shown in Fig. 3 for 2, 3, 4 and 5 spans and for a typical interior bay.

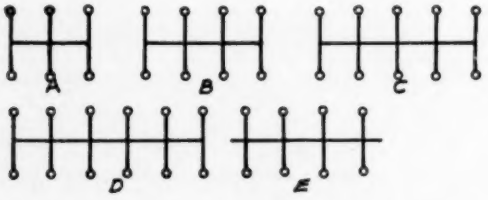


FIG. 3—DIAGRAMS OF FRAMES INCLUDING TWO, THREE, FOUR AND FIVE SPANS, AND TYPICAL INTERIOR BAY

Note the hinged ends of columns for basic frame and see later for use on continuous columns. Each frame has been analyzed for a stiffness ratio of column stiffness I_h/H' to beam stiffness I/L of 0.0 (continuous beams) 0.1, 0.4, 1.0 and 10.0. The resulting

values are given in Table 3 and are also shown in graph form on Figs. 4 to 7 inclusive. The values for $\frac{I_k \cdot L}{H' \cdot I}$ have been drawn on the horizontal axis to a convenient scale. This scale is shown in Table 2, Fig. 2 and its purpose is to make the curves as nearly straight as possible.

Note instructions on Fig. 2 for the use of column stiffness. The stiffness of the exterior columns is frequently different from that of the interior columns. It is recommended that moments for such cases should be adjusted for end bays, using the revised value for column stiffness.

APPLICATION OF THE BASIC FRAME TO CONTINUOUS COLUMNS

The condition as assumed for the basic frame, of column height H' and columns hinged at one end is very rare in building construction. It is necessary to find a relationship between the basic frame and the multiple story building, with story height H.

On Fig. 8A the slope of the elastic curve for a moment $M = 1$ and height H' (the condition similar to the basic frame), is $\phi_1 = \frac{H'}{3EI}$, and on Fig. 8B, the slope for a continuous column (story height H) and for $M = 1$, is

$$\phi_2 = \frac{H}{6EI}\left[3 - \frac{H}{H-B}\right], \text{ for } \phi_1 = \phi_2, \text{ we have:}$$

$$H' = \frac{H}{2}\left[3 - \frac{H}{H-B}\right]$$

The value of B depends on the number of spans, stiffness of columns, etc., but the writer believes that the values given for H'/H in Table 2 are sufficiently accurate for use in practical design. Note that as the story height H is the known value, it will be necessary to make a few trials so that the values for $\frac{I_k \cdot L}{H' \cdot I}$ and H'/H in Table 2 correspond.

The Influence of B Values. Fig. 9 has been prepared to better demonstrate the meaning of B values or H'/H. Although the writer believes that satisfactory results can be reached by using the values H'/H from Table 2 for all types of loadings (for all

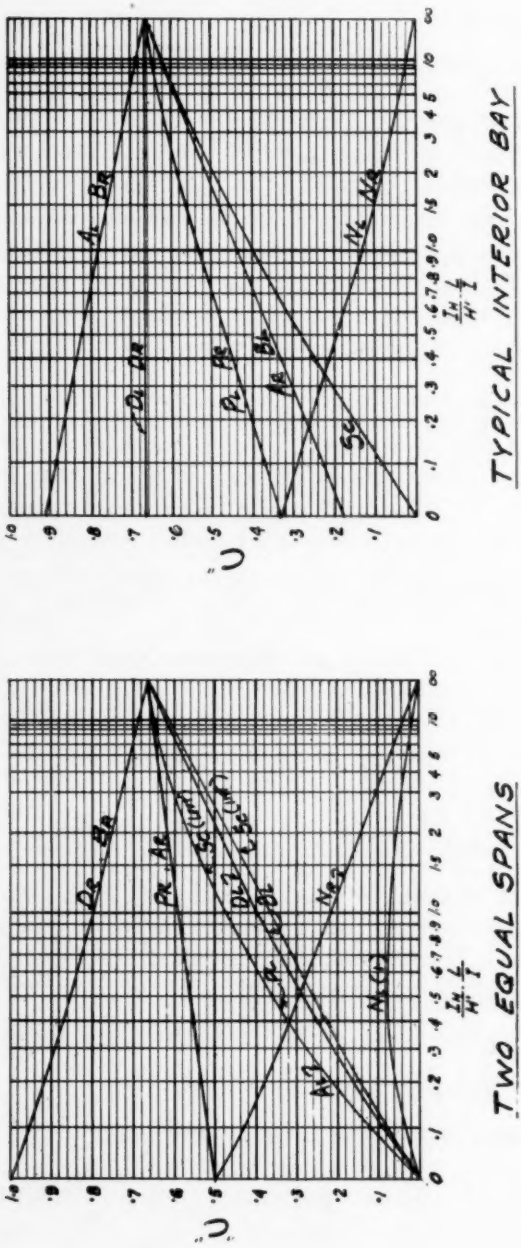


FIG. 4—GRAPH FOR TWO EQUAL SPANS AND TYPICAL INTERIOR BAY

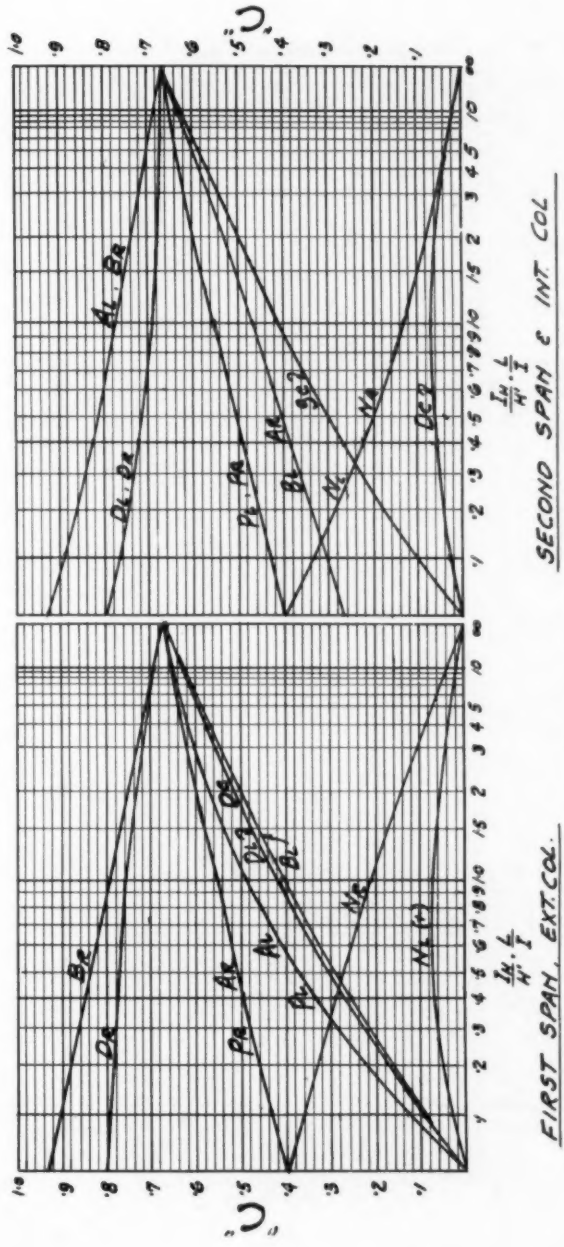


FIG. 5—GRAPH FOR THREE EQUAL SPANS

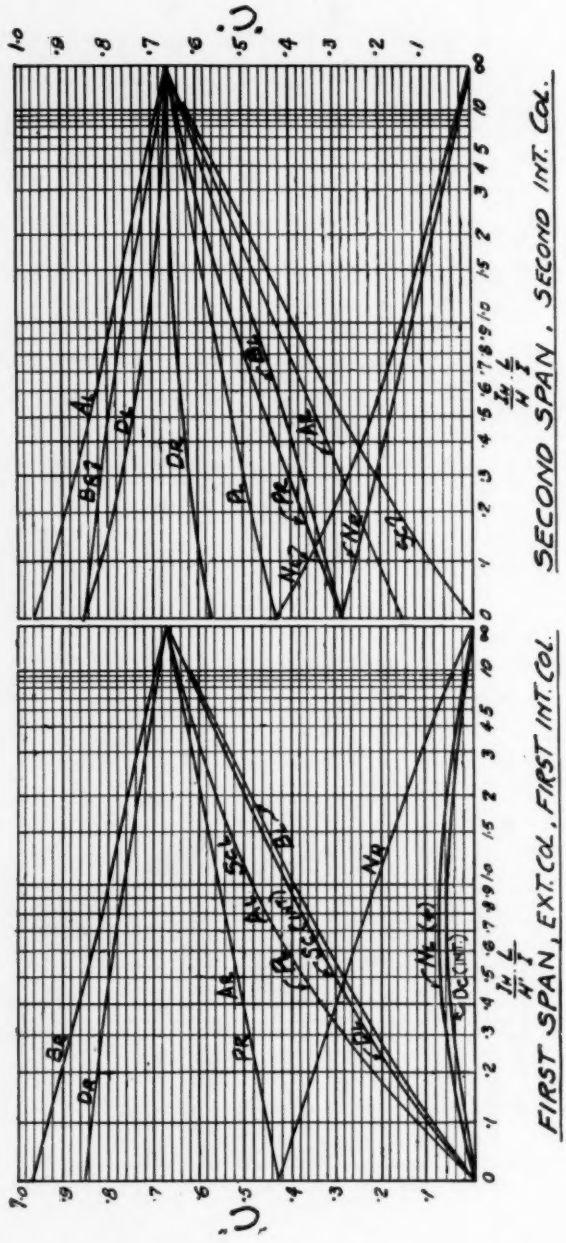


FIG. 6—GRAPH FOR FOUR EQUAL SPANS

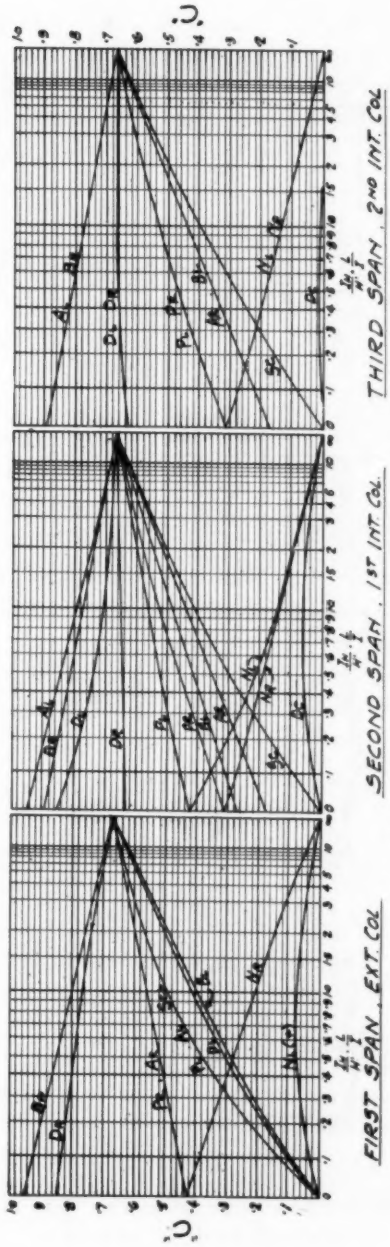


FIG. 7—GRAPH FOR FIVE EQUAL SPANS

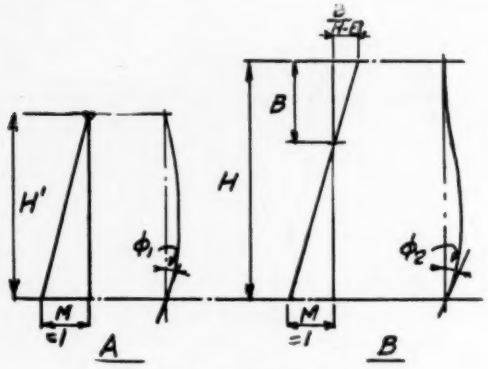


FIG. 8—A AND B

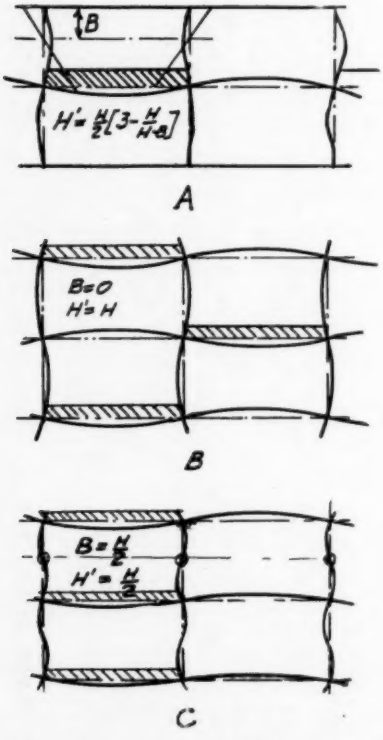


FIG. 9—POINTS OF CONTRAFLEXURE FOR DIFFERENT ARRANGEMENTS OF LOADING

constants), this is not strictly correct, as will be noted from the following: For loading as shown Fig. 9A, typical interior bays, alternately loaded and unloaded (condition of maximum positive moments), and floors over and under uniformly loaded and therefore without effect on columns, the column moment is 0 at a distance B from top, and $H' = \frac{H}{2}\left[3 - \frac{H}{H-B}\right]$

This is the condition as expressed in values for H'/H in Table 2. The condition of loading as shown in Fig. 9B assumes that all stories are alternately loaded and unloaded. The point of contraflexure of the columns occurs at the joint and $B = 0$, $H' = H$. This would give higher values for the positive moments, but as this condition of loading is very unlikely, it is not generally adopted in commercial design.

Points of contraflexure as shown in Fig. 9C would occur when bays are correspondingly loaded and unloaded throughout. $B = H/2$, $H' = \frac{1}{2}H$. This condition of loading is of no practical value except that a similar deformation of columns takes place for dead loads (except for typical interior bays where there is no column bending), making $H' = \frac{1}{2}H$ more accurate in determination of the dead load coefficient. For comparison of results from this method with results derived from the usual application of the slope deflection method, assuming columns fixed at adjacent floors, note that $B = \frac{1}{3}H$, or $H' = \frac{3}{4}H$.

For the practical application of this method, the reader is referred to examples in Figs. 10 and 11.

Side Movements. It should be noted that all floors are assumed to be held in position so that no form of loading can cause any horizontal movement. Dead loads have no such effect, nor does any symmetrical form of live loads. It is only loading not symmetrical about the centre line of the complete frame system that will have any effect, and for many columns in the frame the effect is negligible. For building construction it is fair to assume that not all bays will cause movement in one direction and that the floor will act to even out these tendencies so that they can be neglected.

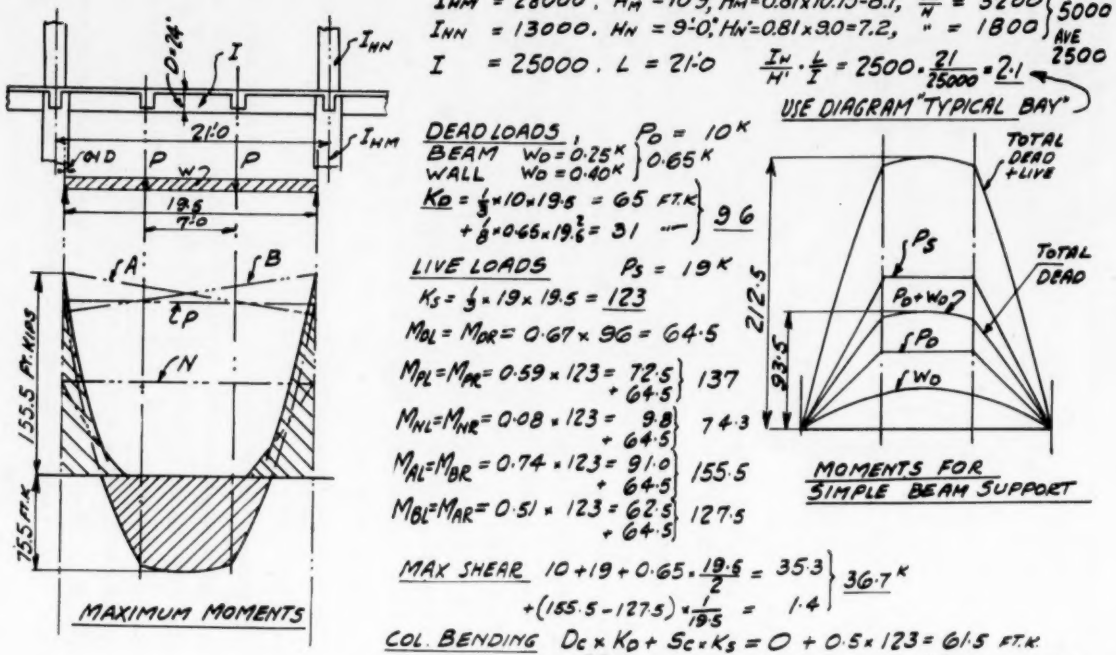
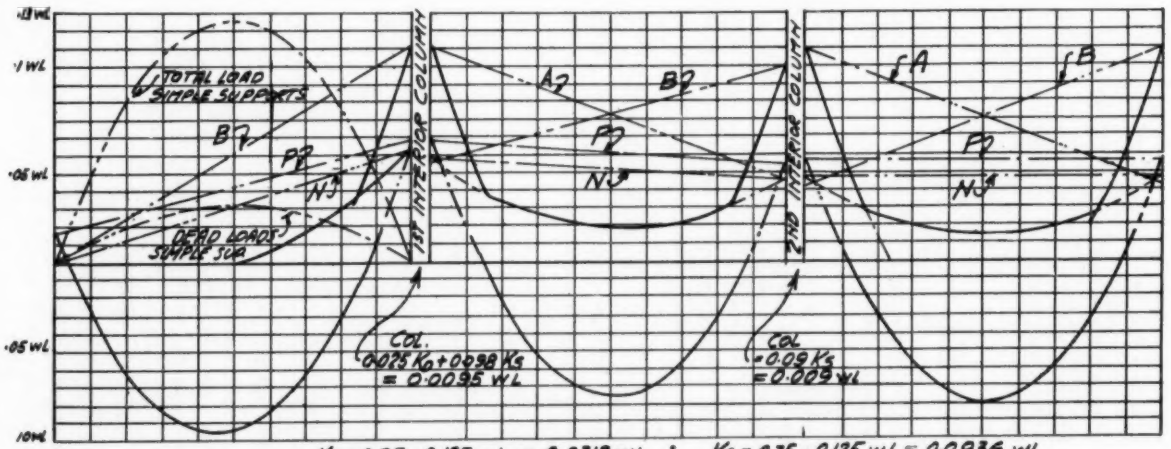

I_HM = 28000 . H_M = 10'9" H_M' = 0.81 x 10.75 = 8.7, I_M/H = 3200
I_HN = 13000 . H_N = 9'-0" H_N' = 0.81 x 9.0 = 7.2, " = 1800
5000 AVE 2500
I = 25000 . L = 21'-0
I_M/H' . L/I = 2500 . 21/25000 = 2.1
USE DIAGRAM "TYPICAL BAY"
DEAD LOADS
BEAM W_D = 0.25^K
WALL W_D = 0.40^K
P_D = 10^K
0.65^K
K_D = 1/3 x 10 x 19.5 = 65 FT.K
+ 1/8 x 0.65 x 19.5^2 = 31
96
LIVE LOADS P_S = 19^K
K_S = 1/3 x 19 x 19.5 = 123
M_DL = M_DR = 0.67 x 96 = 64.5
M_PL = M_PR = 0.59 x 123 = 72.5 + 64.5 = 137
M_NL = M_NR = 0.08 x 123 = 9.8 + 64.5 = 74.3
M_AL = M_BR = 0.74 x 123 = 91.0 + 64.5 = 155.5
M_BL = M_AR = 0.51 x 123 = 62.5 + 64.5 = 127.5
MAX SHEAR 10 + 19 + 0.65 . 19.5/2 = 35.3
+ (155.5 - 127.5) x 1/19.5 = 1.4
36.7^K
COL. BENDING D_C x K_D + S_C x K_S = 0 + 0.5 x 123 = 61.5 FT.K
COL. UNDER 3.2/5.0 x 61.5 = 39.3 FT.K COL OVER 1.8/5.0 . 61.5 = 22.2 FT.K
MAXIMUM MOMENTS
155.5 FT.KIPS
75.5 FT.K
TOTAL DEAD +LIVE
TOTAL DEAD
212.5
93.5
MOMENTS FOR SIMPLE BEAM SUPPORT

FIG. 10—TYPICAL EXAMPLE. BEAM MOMENTS



$K_D = 0.25 \times 0.125\ WL = \underline{0.0312\ WL}$; $K_S = 0.75 \times 0.125\ WL = \underline{0.0936\ WL}$

$M_{DL} = 0.098\ K_D = 0.00306\ WL$
$M_{DR} = 0.829\ K_D = 0.0259\ WL$
$M_{PL} = 0.129\ K_S + 0.00306\ WL = 0.015\ WL$
$M_{PR} = 0.447\ K_S + 0.0259\ WL = 0.068\ WL$
$M_{NL} = +0.032\ K_S + 0.00306\ WL = \sim 0$
$M_{NR} = 0.382\ K_S + 0.0259\ WL = 0.062\ WL$
$M_{AL} = 0.129\ K_S + 0.00306\ WL = 0.015\ WL$
$M_{AR} = 0.447\ K_S + 0.0259\ WL = 0.068\ WL$
$M_{BL} = 0.080\ K_S + 0.00306\ WL = 0.007\ WL$
$M_{BR} = 0.924\ K_S + 0.0259\ WL = 0.112\ WL$

$M_{DL} = 0.804\ K_D = 0.0251\ WL$
$M_{DR} = 0.634\ K_D = 0.0198\ WL$
$M_{PL} = 0.455\ K_S + 0.0251\ WL = 0.068\ WL$
$M_{PR} = 0.358\ K_S + 0.0198\ WL = 0.053\ WL$
$M_{NL} = 0.349\ K_S + 0.0251\ WL = 0.058\ WL$
$M_{NR} = 0.276\ K_S + 0.0198\ WL = 0.046\ WL$
$M_{AL} = 0.918\ K_S + 0.0251\ WL = 0.111\ WL$
$M_{AR} = 0.244\ K_S + 0.0198\ WL = 0.043\ WL$
$M_{BL} = 0.315\ K_S + 0.0251\ WL = 0.055\ WL$
$M_{BR} = 0.862\ K_S + 0.0198\ WL = 0.100\ WL$

$M_{DL} = M_{DR} = 0.641\ K_D = 0.020\ WL$
$M_{PL} = M_{PR} = 0.366\ K_S + 0.020 = 0.0543\ WL$
$M_{NL} = M_{NR} = 0.275\ K_S + 0.020 = 0.045\ WL$
$M_{AL} = M_{BR} = 0.867\ K_S + 0.020 = 0.101\ WL$
$M_{AR} = M_{BL} = 0.222\ K_S + 0.020 = 0.041\ WL$

5 SPANS $\frac{I}{H} = 0.1\frac{I}{L}$
LIVE LOAD = 3 TIMES DEADLOAD

FIG. 11

CONCLUSION

All constants in Table 3 have been figured following the method originally published by A. Strassner in his book "Neuere Methoden zur Statik der Rahmentragwerke." This method eliminates long equations, and any error is very soon discovered.

The writer wishes to acknowledge helpful suggestions received from M. P. van Buren of J. Di Stasio & Co., New York City.

APPENDIX 1

The Strassner method is a continuation of and improvement on the Ritter method. Both methods are based on Mohrs principle "that the ordinates of the elactic line are the bending moments of the beam from a loading at each point equivalent to the moment area divided by $E. I.$"

Strassner derives two points, J and K, for each member of the frame system, J near the left support and K near the right support. The location of these points expresses the combined elastic value (stiffness) of all of the structure that lies respectively at the left of J and at the right of K. In the determination of J enter the values of E, I and L for all members, beams and columns, of the left part of the structure, and for K, in a similar manner, values for the right part.

As the basic frame in this paper has columns hinged at opposite ends, the above mentioned fixed points need only be determined for beams. The general equations are shown in Fig. 12. For the various stiffnesses investigated they are for J points:

$I_H/H^1 \cdot L/I$	a_1	a_2	a_3	a_4	a_5	
0	0.0000	0.2000	0.2105	0.2112	0.2113	$\times L$
0.10	0.0769	0.2170	0.2237	0.2241	0.2241	
0.40	0.1818	0.2480	0.2500	0.2500	0.2500	
1.00	0.2500	0.2758	0.2763	0.2764	0.2764	
10	0.3226	0.3232	0.3232	0.3232	0.3232	

The same values apply to K points, but starting at the extreme right support.

The support moments are figured independently for loading at any one span. A load on span CD, Fig 12, created moments at the J and K points, the values of which are directly proportional to the moments at the support. See Fig. 13.

$$M_J = -\frac{a}{L} \cdot \phi_C \cdot \frac{6 \cdot E \cdot I}{L}; \quad M_K = -\frac{b}{L} \cdot \phi_D \cdot \frac{6 \cdot E \cdot I}{L},$$

where ϕ_C and ϕ_D are the angular deflections from the simply supported load only, at supports C and D respectively. Referring to Table 1 in this paper,

$$\phi_C = \phi_D = \tfrac{1}{3} \cdot K \cdot \frac{L}{E \cdot I} \quad \text{or}$$

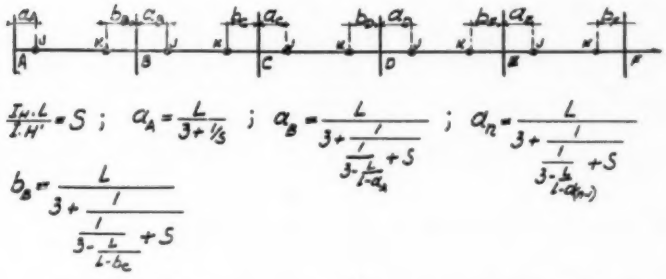


FIG. 12—GENERAL EQUATIONS, STRASSNER METHOD

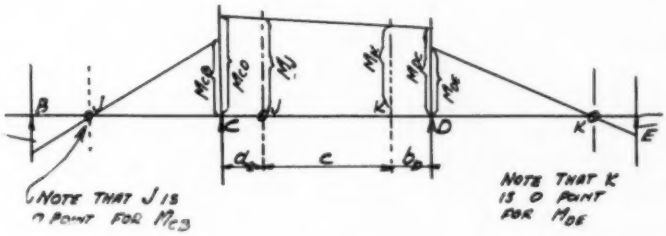


FIG. 13

$$M_C = -\frac{a}{L} \cdot 2K \cdot \frac{L-2b}{c} \quad ; \quad M_D = -\frac{b}{L} \cdot 2K \cdot \frac{L-2a}{c} .$$

Having determined the support moments from loading in any one span, it remains to distribute them between adjacent beams and columns.

From span *CD* in direction *CB*

$$M_{CB} = M_{CD} \cdot \frac{1}{1 + \frac{L}{H'} \cdot \frac{I_H}{I}\left[3 - \frac{L}{L-a_B}\right]}$$

From span *CD* in directon *DE*

$$M_{DE} = M_{DC} \cdot \frac{1}{1 + \frac{L}{H'} \cdot \frac{I_H}{I}\left[3 - \frac{L}{L-b_E}\right]}$$

The differences between the moments at adjacent beam supports are distributed on columns over and under in direct relation to their stiffnesses.

The above mentioned values are, for beams right to left:

$I_B/H^1 \cdot L/I$	*BC* to *BA*	*CD* to *CB*	*DE* to *DC*	*EF* to *ED*
0	1.0000	1.0000	1.0000	1.0000
0.10	0.8392	0.8530	0.8538	0.8538
0.40	0.5844	0.5994	0.6000	0.6000
1.00	0.3750	0.3818	0.3820	0.3820
10	0.0616	0.0616	0.0616	0.0616

Values left to right are determined in a similar manner

APPENDIX 2

The support moments from a concentrated load in any span can be determined (see Fig. 14) as follows:

$$M_L = -\frac{P \cdot X \cdot (L - X)}{L} \cdot \frac{a}{L} \cdot \frac{2L - X - 3b}{C}$$

$$M_R = -\frac{P \cdot X \cdot (L - X)}{L} \cdot \frac{b}{L} \cdot \frac{L + X - 3a}{C}$$

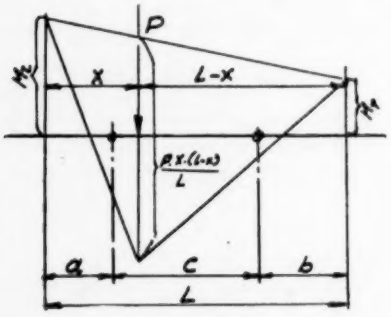


FIG. 14—SUPPORT MOMENTS FOR CONCENTRATED LOAD

These formulas can be used to make allowance for any variation to the assumed symmetrical loading, as the fixed points and distribution of moments can be interpolated from the above tables using the same scale as for the diagrams in this paper.

Readers are referred to the JOURNAL *for April 1931 for discussion which may develop. Such discussion should reach the Secretary by March 1, 1931.*

Discussion of Paper by Willis A. Slater and Inge Lyse

"Compressive Strength of Concrete in Flexure—As Determined from Tests of Reinforced Beams"*

BY FRITZ EMPERGER†

The relation between the compressive strength of concrete in flexure and that in direct compression has recently received increased interest as the methods of designing concrete of a given strength have become better known, and thereby it has become advisable to use lower factors of safety.

In this connection it is not sufficient to have a value like the cube strength which never appears alone in a structure. We have on one side to find the correct column strength, and on the other side the compressive strength in beams. With respect to the columns it is considered established that the column strength is 80 per cent of the cube strength. The question involved is, therefore, to find the multiplication factor for flexural compressive strength.

As a result of my proposal in *Beton und Eisen*, 1903, I tried in 1906, to introduce the use of heavily reinforced concrete beams as control specimens in the construction field, and was able to establish its use at a number of places, as for instance, in the construction of the Building for the Department of War in Vienna. The beam used had a cross-section 70 by 200 mm. (2¾ by 4 in.). The cross-section was later changed to 70 by 96 mm. on account of supporting the steel directly on the form. The loading arrangement was selected such that by the use of a value of n equal to 15, the flexural compressive strength is equal to $P/3$ without further computations.

An investigation by the Austrian Committee for Reinforced Concrete was undertaken in the same year for the purpose of

*A. C. I. Journal, June 1930; *Proceedings*, Vol. 23, p. 831.
†Vienna, Austria.

studying the proper shape and multiplication factor for such a test beam. The given dimensions resulted from the effort at reducing both dimensions and load to such a degree that the beam would be easily handled and could be tested without a testing machine. The studies by the Austrian Committee for Reinforced Concrete included more than 400 beams and a similar number of cubes. The first tests were carried out with fixed beams[1] and later a special series of tests was made.

The voluminous report[2] of these tests was not published until 1916 on account of the war. This discusses different concrete mixtures, beam cross-sections, and reinforcements. The reinforcement used in the beams amounted to 2.02, 4.04, 4.24, 4.97, and 5.83 per cent. Beams of different widths and depths were tested which were constructed according to field methods, as these tests were primarily intended for use on the job. In spite of this fact, the beams showed almost the same variation from the average as that obtained by the tests of the carefully made cubes. The maximum variations for the beams were +19.2 per cent and —16.8 per cent, and for the cubes +13 per cent and —18.2 per cent. The strength of the concrete used was as high as 420 kg. per sq. cm. (6000 lb. per sq. in.). The reinforcement was ordinary steel with a yield point stress of 2400 kg. per sq. cm. (34,000 lb. per sq. in.). It was established that for the ordinary strengths used in constructions, a reinforcement of 4 per cent was sufficient for the concrete in the beam. A width of 7 cm. was considered sufficient to accommodate the maximum size of aggregates used in reinforced concrete construction, since the use of these beams was not considered for large size aggregates.

In the course of progress of these tests, the German Committee for Reinforced Concrete also raised the same question, and received accurate information from us regarding the progress of the work.

The German tests are reported by Bach in Heft 19 of the Committee's publications. The beams were of larger width and had shear reinforcement, and were loaded at the center. As the

[1] "Berichte des Oesterreichischen Eisenbetonausschusses" Heft 4 by Dr. Fritz Emperger.
[2] "Berichte des Oesterreichischen Eisenbetonausschusses" Heft 6 by Hofrat Nähr.

latter method gives only the results for a narrow element across the beam it was soon abandoned.

The publication had the result that many doubts were raised in the German technical literature, while in Austria it had already been decided to adopt the beam test in the specifications. A second publication by Dr. Petry in Heft 50 of the reports from the German Committee for Reinforced Concrete, met the various objections and also showed that the cubes could not be made reliable on the job while the test beam could. Finally, the test beams have been included in the German specifications of 1926, and in the later specifications for concrete control, and at the same time a handy testing machine for the observations of maximum loads has been developed.

Before we discuss the important relationship between cubes and beams, we will study some American tests. The first publication was by Slater and Zipprodt in the *Proceedings* of the American Concrete Institute for 1920,[3] and then by Slater and Lyse in the American Concrete Institute's JOURNAL for June, 1930. It should be first mentioned, that the American engineers have changed from the use of the cube. It was first established in Germany that the cube strength is not even present in the columns and that the prism strength or actual strength of the concrete is about 80 per cent of the cube strength. No conclusion, however, has been drawn from these results, and the cube form has remained an artificial measure of the strength and other properties of the concrete. In North America, however, a cylindrical form of 15 cm. diameter and 30 cm. height has been adopted and this specimen gives numerous advantages over the cubes. It is always easy to make the cylinder mold from a sheet of galvanized iron, and glass plates will serve as end plates. It is of special importance to have a uniform method of filling and this cannot be secured in the corners of the cubes. The same difficulty is present in the capping of the cubes which might produce uneven load distribution. It seems probable that the cylinder sooner or later will be adopted also in Europe. The 30-cm. cubes give 13 per cent higher strength than the cylinder, the 20-cm. cubes give

[3] "Compressive Strength of Concrete in Flexure," by W. A. Slater and R. R. Zipprodt, A. C. I. *Proceedings*, Vol. 16, p. 120.

20 per cent higher strength, and the cylinder strength can be considered equal to the columns. (In order not to confuse the reader, we will use the 20-cm. cube strength throughout the rest of this discussion.)

It is of especial interest to note that the American tests included a further form of flexural strength in its program. This corresponded to the important transition form between pure compression and pure bending by eccentric load on compression specimens, which results in combined bending and direct compression. We see from these tests that the actual distribution of stresses does not correspond completely to that given by the assumption of Hooke's law, and that the error is due to this assumption. The eccentric tests have shown that at maximum load the compressive strength is 20 per cent higher than the cylinder strength, even though the relation between base and height was the same in both cases. In this case the load does not correspond to the ordinary column strength, but to the 20 per cent higher cube strength.

Further studies by Slater and Lyse bring forth that the ratio decreases as the strength of the concrete increases, so that the leaner concrete shows a relatively higher flexural strength than the richer concrete. This is a new proof of the fact that this concrete contains strength properties which should not be underestimated and which is very useful where the structure is expected to deform under secondary stresses.

When we consider the question of multiplication factor it must be remembered that the Austrian specifications assume 1.33, and the German tests show 1.70 and higher. A superficial reader will find a great difference between the results for the German and the Austrian tests. As the Austrian tests and original reports are very little known, it is quite regrettable that the German writers have not taken notice of them. The fact is that the German figure is based on the 30-cm. cube and the beam loaded at the center. According to the limited tests in Heft 19 "Deutsche Ausschuss für Eisenbeton" we find that this arrangement gives 9 per cent too high a value. We have, therefore, first to subtract from 1.7, 9 per cent due to the center load, and then 7 per cent due to the use of the 30-cm. cube when we want to obtain the relation for the 20-cm. cube. We find then, that the

German tests give a result of 1.45 as multiplication factor, or only slightly different from the specified Austrian value. The Austrian value did not correspond to the tests made, but was purposely chosen somewhat lower. The average of the tests in both countries gave approximately the same value. The question arises why the American tests give so much higher values. These tests may not be convincing because they were obtained on a small number of specimens, as also some of the Austrian tests showed higher values in some of the groups of tests. In order to arrive at a useful average, a much larger number of tests and also tests including different aggregates should be made, as the American tests only included one kind of aggregate. In order to obtain a complete view of the situation and summarize the differences obtained, it may be said that the multiplication factor given by the Austrian specifications is too low and instead of 1.33 the number 1.5 corresponds more closely to the actual relation. Otherwise a too high cube strength would be derived from the control beam. One thing should not be overlooked, and that is that the cube strength should not be considered fixed and constant, and the flexural strength liable to large variations. The situation is more clear when we disregard the imagined strength and judge the case by the deformation at failure, and keep in view the fact that the failure neither by cubes nor by beams, is caused by compression, but by the shear stresses which cause the deformation at failure. We will then see that the deformation at failure depends upon the whole set of conditions, that is, from the dimensions of the specimens to the quality of the concrete, and that the ratio is subject to large variations, and cannot be given a fixed value. When, however, we use such a ratio it is necessary to fix the limits which will be within the given factor of safety and not let the selection of the ratio depend on anyone's judgment. The increase in compressive strength obtained by eccentric loading is important when the eccentric load has been neglected in the design of structure. In the building of arch bridges where this should be done with great accuracy, this increase is found to amount to 20 to 50 per cent and increases with the eccentricity.

How sensitive the beam strength is with respect to all kinds of effects is brought forth in my report in *Forscherheft*, 14, 1911,

which contains the use of a strength thermometer by which it was possible to conclude what temperature was present at the time of set of the concrete from the strength results of the test beams.

AUTHORS' CLOSURE

The authors are glad that Dr. Emperger has given some historical information on the Austrian and German investigations designed to determine the compressive strength of concrete in flexure and of the use of small heavily reinforced beams as control specimens. Certain advantages of this form of control specimen are recognized. Its principal disadvantage probably lies in the fact that it is more difficult and expensive to make and test this control beam than it is to make and test a small cylinder or a cube. Due to this fact it would not be feasible to provide control tests from as many batches of concrete as may be sampled for a given cost by means of small cylinders or cubes.

Dr. Emperger indicates that the Austrian values are considerably lower than those given in this paper. Unfortunately we have not been able to compare his results with ours. Finality is not claimed for the results of our tests. Nevertheless, every reasonable effort was made to keep the tests free from effects which would vitiate the results. It is recognized that much more ground could be covered in an investigation which included 400 instead of 36 beams. However, three beams of each kind were made and the close agreement between results for individual beams should be evidence that the work was carefully done. In the absence of any suggestion as to any features in which the methods used were defective, the authors are inclined to believe that the results may safely be applied within the field which the investigation covered.

It is quite possible that if long-time loads had been applied to the beams, lower strengths would have resulted. However, the same is true regarding the cylinder tests, and there is no certainty that the ratio of the compressive strength in flexure to that in direct compression would have been any lower than those found, if all the tests had been made under long-time loading. So far as the authors know, this is the most important feature not covered in the investigation.

An effort was made to review all the previous literature on the subject under investigation, and it is regretted that the tests carried out by Dr. Emperger were not found in the search that was made. His discussion of this paper is highly appreciated.

The authors wish to correct an error which has been pointed out in Table 7 of the paper under discussion. The computed stresses at failure, based upon the observed values, of n as given in Column 6 and the resulting beam-cylinder ratios as given in Column 9 of Table 7 are both somewhat too small. The correct values are:

Column 6	*Column 9*
2340	1.68
3130	1.12
4250	1.04
5050	1.05
5680	.99
3670	1.42
4140	1.00
3550	1.20
3460	1.25
3570	1.23
3080	1.09
4120	1.08

The error is deeply regretted, but fortunately it is not of sufficient magnitude to affect any conclusions in the paper. Even the curves in Figures 21 and 22 are not affected appreciably by the error.

ABSTRACTS

ABSTRACTS

of the current literature of cement and concrete

PREPARED UNDER THE DIRECTION OF THE
AMERICAN CONCRETE INSTITUTE
COMMITTEE ON ABSTRACTS

J. C. WITT, Chairman, (*Materials*)
H. F. GONNERMAN (*Properties of Concrete*)
F. E. RICHART (*Engineering Design*)
REXFORD NEWCOMB (*Architectural Design*)
THEODORE CRANE (*Field Construction*)
W. D. M. ALLAN (*Shop Manufacture*)

EDITED BY DWIGHT F. JENNINGS

ABSTRACTORS

JOHN E. ADAMS
F. O. ANDEREGG
ROBERT W. ABBETT
ODD ALBERT
A. G. ALEXANDER
S. O. ASPLUND
C. BACHMANN
P. H. BATES
A. A. BRIELMAIER
A. E. BEITLICH
MILES N. CLAIR
C. G. CLAIR
IRA L. COLLIER
M. A. CORBIN
E. L. ERIKSEN
H. FRAUENFELDER
W. I. FREEL
P. J. FREEMAN
H. J. GILKEY
A. T. GOLDBECK
E. S. HANSON
S. C. HOLLISTER
A. J. HOSKIN
F. H. JACKSON
V. P. JENSEN
D. E. LARSON
H. WALTER LEAVITT
L. G. LENHARDT
A. E. LINDAU
ARTHUR R. LORD
H. J. LOVE
P. MCKIM
JOSEPH MARIN
N. D. MITCHELL
REXFORD NEWCOMB
J. C. PEARSON
C. E. PROUDLEY
F. E. RICHART
N. H. ROY
J. H. SHANK
EDMUND SHAW
A. SIZE
T. U. TAYLOR
T. THORVALDSON
G. E. TROXELL
H. S. VAN SCOYOC
STANTON WALKER
G. M. WILLIAMS
C. R. YOUNG
ROY N. YOUNG

MATERIALS

ADMIXTURES

Diatomite as concrete admixture material. ELLIOTT S. HASTINGS. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 55.—Representative diatomites were selected for a series of tests, all other materials being uniform. It was found that increase in compressive strength varied with alumina and iron oxide contents of the diatomites, a table showing increases from 16 per cent to 28 per cent in compressive strength with additions of 3 per cent diatomite which varied from 2.40 per cent to 17.86 per cent alumina and iron oxide. Author says he has never found a loss of strength from addition of any diatomite, all cases reported being found due to conditions of mixing and pouring. Reduction of laitance and efflorescence has been found proportional to silica present. Amorphous silica present presumably combines with lime liberated from cement thus reducing efflorescence, and absorptive power of diatomite prevents laitance. There appears to be a definite relation between lubricative effect, or workability added by diatomite, and shape of particles. Photomicrographs and chemical analysis are given for various dolomites used in the strength tests. It is considered a matter of economics whether the desired strength should be secured by the use of the proper amount and type of diatomite or regulation of the water-cement ratio.—EDMUND SHAW

AGGREGATES

Breeze and clinker concrete. *Engineering* (England), July 11, 1930, V. 130, No. 3365, p. 41.—The British Department of Scientific and Industrial Research has issued Special Report No. 15 dealing with corrosion of steel by breeze and clinker concrete. It was found, generally speaking, the corrosive power of concrete increases with increasing sulphur content of the aggregates, and that sometimes even a small amount of sulphur left in well vitrified clinker will provide rusting. The extent of this action is reduced when fines in clinker aggregate is replaced by sand. Rusting may occur under dry indoor conditions. It is recommended that safe practice requires the use of coal residue of any kind as aggregates to be forbidden in concrete if the material is to come in contact with steel when in use.—G. M. WILLIAMS

Use of sand from dunes for concrete structures in sea water. A. GUTTMANN. *Bautenschutz* (Germany), March 20, 1930, V. 1, No. 1, p. 6-10.—Fact that many failures of concrete structures in sea water occurred when sand from dunes was used, was investigated and it was found that sand from dunes has 26.1 per cent more voids than normal river sand. Strength tests were made with regular and dune sands. In order to obtain same density with dune sand, 23.6 per cent more cement was required. Dune sand is suitable for sea water structures, when used in rich mixtures. Latter can be made normal, when concrete is allowed to harden in air before placing in water.—A. E. BEITLICH

The bulking properties of microscopic particles. PAUL S. ROLLER. *Ind. Eng. Chem.*, Nov., 1930, V. 22, No. 11, p. 1206-8.—By means of an air analyzer for fine powders, an anhydrite, gypsum, portland cement, and chrome-yellow powder were separated into homogeneous fractions above 1 micron surface mean diameter. The bulkiness, or volume per unit weight, was found to increase with decrease in particle size below a critical diameter; for particle sizes greater than the critical diameter bulkiness was constant. For all powders the same functional relationship was found to hold between voids per gram and surface mean diameter. The differences in the value of the constants for the different powders appear to depend chiefly on the particle shape, whether

cubic, prismatic, basal, etc. As would be expected from a consideration of packing of small particles in voids of larger particles, surface mean diameters of portland cement and chrome-yellow microscopic powders calculated from the bulkiness of the powder were found to average several tenths of a micron higher than true mean diameters. With this correction in mind, for powders above 1 micron surface mean diameter, a rapid method presents itself of estimating the mean particle size of the microscopic powder from a knowledge of its curve of bulkiness vs. particle diameter, but no knowledge as to distribution of particle sizes in the powder is thus obtained.—ROY N. YOUNG

Flexibility. Aim of this crushed stone operation. *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 33.—New plant of the Elmhurst-Chicago Stone Co. replaces two old plants. Outstanding features are: A carefully worked out spout arrangement permitting any mixture of sizes or sending sizes to be recrushed; the use of vibrating screens for both scalping and sizing; the use of conveyors instead of bucket elevators throughout; a surge or equalizing bin and a variable speed feeder to control the flow of the stone to the screening plant; starting and stopping of all screen house equipment from one of several conveniently located push buttons; safety device in surge bin to give warning when the bin is nearly full and to stop conveyor when it is full. Only steel and concrete are used in construction. A concrete products plant is operated and the office and some other buildings are made from its product. An unusually complete machine shop is included.—EDMUND SHAW

Aboard a lake-going sand and gravel plant. *Rock Products*, Sept. 13, 1930, V. 33, No. 19, p. 41.—Sand and gravel dredge Brazil of the Marine Dredge and Gravel Co., Chicago, makes sand for fine concrete aggregate by a double classification. The sand is pumped by two 18-in. pumps, each supplying its own screening and classifying unit. Stream goes first to a large tank classifier which overflows about one-half of the solids, which is unwanted fine sand. After passing a screen to take out gravel the sand and water go to a surface current classifier with three spigots. These discharge three products, coarse, medium and fine. Any part or all of any one of these may be wasted and by mixing the required amount of each the grading may be kept constant. —EDMUND SHAW

Standardizing aggregate sizes in Los Angeles. HARRY D. JUMPER. *Rock Products*, April 12, 1930, V. 33, No. 8, p. 65.—As many as 22 sized aggregate products were formerly sold in Los Angeles, each having a different trade name. After a merger of larger companies 11 products were made standard. The aggregates used for cement concrete have been designated as "gravels" and the aggregates for bituminous pavements as "crushed rock" although both are largely crushed gravel and boulders. The sizes for concrete range from No. 1, 2½ to 1½-in. to No. 4, ½-in. to 10 mesh. Gravel No. 123 is a mixture of sizes from 2½-in. to ¼-in. This is not used as much as formerly, practice being to deliver separated sizes to job, thus avoiding segregation.—EDMUND SHAW

An independent study of aggregate characteristics. LEROY C. GILBERT AND H. F. KRIEGIE. *Rock Products*, June 21, 1930, V. 33, No. 13, p. 58.—Following their investigations on gradation and voidage these authors describe an investigation into comparative merits of gravel, stone and slag as concrete aggregates. Investigation takes in not only their own tests but those of Kellerman of the Bureau of Public Roads and of Walker and Proudley. Mixes were made for a constant mortar-voids ratio and cement content was checked by analysis. Differences in compressive and transverse strength were finally ascribed to poor gradation, the weak concrete being deficient in the ¼-in. to ¾-in. particles. This was checked by specimens which had these sizes in

excess and those having just the amount to make a smooth distribution curve. The distribution curve is a form of direct plot of sieve tests in which each sieve is represented by a vertical line. The height of a point on any line represents the percentage remaining on the sieve it represents. Points are connected to form a curve. Horizontal spaces between the vertical lines have no value. Gap gradings and excess of any size show up better than with other plots. The authors conclude that any notable differences in the concrete making quality of the aggregates tested must be ascribed to inferior gradation where they cannot be ascribed to differences in cement content or mortar-voids ratio. It is suggested that different gradations of sand might sometimes be specified for stone than for gravel, the stone (or slag) with high voids requiring a coarser sand. The most efficient combined aggregate was found to have from 21 per cent to 29 per cent of 10-mesh to ¾-in. material. Differences of 10 per cent in strength can result more easily from changed gradation than from a change from one type of aggregate to another. Until our present conception of fine aggregate as ¼-in. to 0 changes we must supply the intermediate material with fine material in the coarse aggregate.—EDMUND SHAW

The missing link in aggregates. LEROY C. GILBERT AND H. F. KRIEGE. *Rock Products*, June 21, 1930, V. 33, No. 13, p. 55.—The authors review the work of Goldbeck and Smyers on the relation of gradation to voids and carry it further in their own investigations. They show the relation to be: (1) The more nearly uniform the size of particles in an aggregate, the greater is the voidage. (2) When two aggregates of the same general shapes but of different sizes are combined in several proportions, the greatest void reduction occurs with approximately 50-50 per cent combinations. (3) The more widely the two fractions to be combined differ in size, the greater is the void reduction. Thus a mixture of approximately 50 per cent 2-in., 1½-in. aggregate with 50 per cent below 200-mesh material is the densest observed mixture of any two separates between these two extremes. (4) As the "gap" between coarse and fine aggregates is increased, voidage of each portion increases while voidage of combined portions decreases. Thus the separate portions 2 in., 1 in. and 28 to 100-mesh particles have higher voidage than 2 in., ½ in. and 8 to 200-mesh particles respectively. However, the voidage of the 2 in., 1 in. and 28 to 200-mesh sizes combined is less than that of the 2 in., ½ in. and 8 to 200-mesh sizes combined. (5) When more than two fractions are mixed lowest voidage results from combination which provided sizes of particles just fitting into the voids of each larger fraction or combination of fractions. With spheres of unit diameter the next smaller separate which can be used to just fit without crowding into the interstices is 0.414 units in diameter. Authors conclude that low density mixtures are not best aggregates on account of their "gap grading." Most complete aggregates are short in ½ in. to ¾ in. size.—EDMUND SHAW

Promotion of premium aggregates. STANLEY M. HARDS. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 70.—Opinion is expressed that engineers are beginning to discriminate among concrete aggregates, and producers will do well to investigate their products and reprocess if necessary. Adoption of trial method of designing mixes encourages the practice of discrimination. The author gives California highway practice in the design of concrete mixes. Trial mixes are made with a definite oversanding of the voids in the coarse aggregate, the cement being fixed. Weight proportions are converted by volume to give yield. With the cement and water fixed, workability can be secured by changing the sand grading, by changing the sand proportion and by changing grading and proportion. Experience shows that sand for maximum workability with minimum water ratio will vary from 30 per cent to 40 per cent of total aggregates. Sand characteristics influence water more than coarse aggregates but both should be considered together. There is little difference in amount of oversanding required with two different coarse aggregates if these

have same voids, but if two different sands are used with the same coarse aggregate the amounts required to give the same workability will vary widely. Field practice indicates need of studying grading curves on each side of division between fine and coarse aggregates or any two ranges of sizes that may be furnished for blends to insure that sizes around the division are not deficient. The sizes from ¾ in. to 10-mesh are important not only to secure strength but because they influence the mixing time. High end fraction gradings may have low voids but they need twice the mixing time of mixes with intermediate sizes. About 50 per cent of the mix should be intermediate sizes.—EDMUND SHAW

Effect of grading of gravel and sand on voids and weights. STANTON WALKER. *Nat. Sand Gravel Bul.*, Sept., 1930, V. 11, No. 9, p. 7-10.—Report of tests in Research Laboratory of National Sand and Gravel Association in 1928 on one gravel and two sands for a wide range of gradings, as preliminary studies of gravel ballast, voids in different combinations of sand and gravel and the effect of these combinations on the strength, yield and workability of concrete. Tests were on gravel separated into the following four sizes: No. 8 to No. 4, No. 4 to ⅜-in., ⅜ to ¾-in., ¾ to 1½-in., and recombined into 57 different gradings. Most of the tests were made with the three coarser sizes; the combinations were selected so that the voids and unit weights may be estimated by interpolation for any combination of the three sizes and a limited number of the four sizes. Each of the two sands was separated into three different sizes: Fine (approximately 0 to No. 48), medium (approximately No. 48 to No. 14) and coarse (approximately No. 14 to No. 4), and were recombined into 51 different gradings, arranged in such a manner that voids and unit weights may be estimated for any other combination of them. Weights per cu. ft. were determined in cylindrical measures with height approximately equal to diameter. A measure of ½ cu. ft. capacity was used for gravel tests and of ⅛ cu. ft. for sand tests. Tests were made for two methods of filling the measure—"dry and loose" and "dry and rodded." Each value is the average of two or three tests made on each grading. While report does not pretend to discuss *effect* of voids on suitability of sand and gravel for various purposes, it does contain valuable and comprehensive information on the relationship between grading of sand and gravel and its void content. Following observations from the tests are given as a guide in studies of the data: (1) For three sizes of sand or gravel, such as used in these tests, least voids were obtained for a combination of fine and coarse sizes, with intermediate size omitted. In general about 30 to 40 per cent of finer size gave least voids. (2) Combinations of two adjacent sizes, that is, fine and medium or medium and coarse showed relatively small changes in void content for different proportions. However, as in the case of fine and coarse, a combination of adjacent sizes containing about 30 to 40 per cent of the finer size had a lower void content than other combinations of two adjacent sizes. (3) General observation may be made that, for sizes used in these tests, void content of any size was reduced by addition of a finer size up to about 40 per cent or of a coarser size up to about 60 per cent. (4) There was an approximately constant difference between the voids as determined by the loose or rodded method. In case of gravel this difference was 3 to 4 per cent and in case of sand it was 5 to 7 per cent.—P. MCKIM

Effect of grading of sand and coarse aggregate on voids and weight of mixed aggregate. STANTON WALKER. *Nat. Sand Gravel Bul.*, Oct., 1930, V. 11, No. 10, p. 1-10.—These data are from hitherto unpublished tests of the Structural Materials Research Laboratory, Lewis Institute, Chicago, released for this purpose by F. R. McMillan, Director of Research of the Portland Cement Association. Complete significance of the test results cannot be considered without taking into account effect of voids on whatever finished product utilizes the mixtures of fine and coarse material. Following indications

of relationships between grading and voids seem clear from the data and should be of interest: (1) For quantities of sand greater than required to produce minimum voids the mixtures containing coarser sand produced lower void contents. (2) Coarser sands used in this investigation had lower void contents than finer ones. (3) For gravel as coarse aggregate same percentage of sand produced minimum voids, regardless of grading of sand; the minimum voids obtained with a given grading of gravel was same for different gradings of sand. (4) For granite as coarse aggregate, percentage of sand required to produce minimum voids varied as grading of sand varied; there is, however, no indication of a definite relationship. (5) Percentage of minimum voids for each combination varied with the grading of the coarse aggregate. The larger sized coarse aggregates gave the lower void contents for mixtures of sand and coarse aggregate. This relationship seems to be a function of size of particle and, probably, of size of void spaces rather than percentage of voids in coarse aggregate.—P. McKIM

Effect of moisture content on voids in sand. C. E. PROUDLEY. *Nat. Sand Gravel Bul.*, Oct., 1930, V. 11, No. 10, p. 10-12.—Free moisture in fine aggregate causes a phenomenon known as bulking, which has been the cause of considerable difficulty in undersanded mixes and over-run in cement when volumetric proportions did not take it into account. Weight proportioning has overcome much of difficulty, but where volume batching is used it is of great importance that bulking characteristics of sand be known. From study of several investigations carried out by the Structural Materials Research Laboratory of Lewis Institute, the Kansas State Highway Laboratory, and the Bureau of Public Roads, the following conclusions have been drawn: (1) Sands increase in bulk when moist. Amount of bulking depends upon percentage of moisture and method of measurement. Bulking is accompanied by increase in percentage of voids. (2) Maximum bulking is obtained with 3 to 10 per cent moisture, depending on grading of sand. Finer sands require more moisture to produce maximum bulking than do coarse sands. (3) Fine sands bulk more than coarse sands. For the usual conditions (3 to 6 per cent moisture) 20 to 45 per cent bulking may be expected depending on grading of sand and method of measurement. (4) Bulking is not a factor in measurement of sand by weight. Weight measurements, however, should take into account weight of water present at time of weighing. (5) Inundation minimizes the bulking of moist sand and gives a value approximately equal to that of the sand measured in a dry and loose condition.—P. McKIM

Proposed German standard specifications for light cast stones made from pumice. (See SHOP MANUFACTURE.)

New forming methods for small concrete buildings. (See FIELD CONSTRUCTION—Buildings.)

CEMENT

Calculation of raw mixture. A. B. HELBIG. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 919-20.—A method is given for calculating necessary amount of raw mixture to produce a certain amount of cement. Calculations are based on wet and dry processes.—A. E. BEITLICH

Tricalcium silicate. A. GUTTMANN AND F. GILLE. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 914-5.—Methods of Hansen and W. Dyckerhoff for preparation of tricalcium silicate are described; and explanation is given for the fact that R. Brill was unable to detect characteristic lines of this compound in his X-ray examinations. (cf. R. Brill, *Zement* (Germany), Aug. 21, 1930, V. 19, No. 34, p. 796 and Jaenicke, *Zement* (Germany), Nov. 21 and 28, 1929, V. 18, No. 47-48, p. 1345-7, 1388-9.)—A. E. BEITLICH

Influence of the monocalcium phosphate on the hydration of cement. KATSUZO KOYANAGI. *Kogyo Kwagaku Zasshi* (*J. Soc. of Chem. Ind.* (Japan), July, 1930, V. 33, Supplemental binding, No. 7, p. 276-7.—It was found by microscopic and physico-chemical methods that very small quantities of monocalcium phosphate retard setting of cement without considerably influencing hardening process. This phosphate has tendency to form insoluble di- or triphosphates with calcium hydroxide, which is liberated by hydration process. Hydration of calcium aluminate seems to be especially effected by above mentioned phosphate.—A. E. BEITLICH

Temperature in the mill and setting time of cements. KATSUZO KOYANAGI. *Zement* (Germany), Oct. 16, 1930, V. 19, No. 42, p. 988-9.—Reply to article written by Heiser (cf. *Zement* (Germany), July 17, 1930, V. 19, No. 29, p. 679-80.) Author describes a number of grinding tests carried out in regular cement mill and in laboratory mill. In general, cements ground in laboratory mill show slower setting properties. Second investigation deals with influence of gypsum on setting time. It was found, that gypsum loses some of its retarding properties, when its percentage in clinker exceeds a certain limit, which depends on chemical composition of clinker. Cements with high SO_3 contents become quick setting when exposed to high temperatures in mill, while cements with low SO_3 contents remain normal.—A. E. BEITLICH

Free lime in portland cement and soundness. GUSTAV HAEGERMANN. *Zement* (Germany), Oct. 16, 1930, V. 19, No. 42, p. 982-4.—Twelve different technical raw mixtures were burned in experimental furnace under same conditions at temperatures ranging from 2237° to 2687° F. Obtained materials were carefully ground (to prevent action of carbon dioxide and water of air) and tested for uncombined lime with method by Emley and for soundness with steam and cold water test. It was found that under mill conditions a good clinker can be produced at 2327° F. Based on steam tests, range of burning temperature is within 392°; based on cold water tests it is within 212° F. Steam test can give good results with clinker with up to 2 per cent uncombined lime (generally only 1 per cent), and clinker with up to 4 per cent can stand cold water test.—A. E. BEITLICH

Estimation of lime in cement. *Engineering* (England), Sept. 5, 1930, V. 130, No. 3373, p. 314.—British Department of Scientific and Industrial Research has issued Building Research, Tech. Paper No. 9, The Estimation of Free Calcium Hydroxide in Set Cement: A Calorimetric Method. The method depends upon the fact that, within small and ascertained limits, the dissociation of calcium hydroxide begins at about 350° C. and is nearly complete after ignition at 550° C. for half an hour. If therefore two parts of a sample of material containing calcium hydroxide are first heated to these temperatures for half an hour and the resulting products are then hydrated, the difference between the two heats of hydration will be a measure of the calcium hydroxide present in the sample providing no other change has been produced in the process. The aggregate error in applying the method to portland cement should not exceed .5 per cent by weight of the sample in the value of the free lime obtained.—G. M. WILLIAMS

What causes the disturbing influence of portland cement on high alumina cement? KOGAKUSCHI AND KATSUZO KOYANAGI. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 866-9.—When portland cement is mixed with high alumina cement, setting process of the latter is greatly accelerated. This fact was investigated by chemical methods and under the microscope. Setting time is lowered very much for mixtures up to 70 per cent portland cement; mixtures with more than 70 per cent portland cement approach normal setting properties. Degree of burning of clinker influences set; clinker with high free lime content causes quick setting. Great acceleration

takes place when high alumina cement is mixed with lime solutions, even 0.5 per cent CaO in solution has great effect. Reactions of high alumina cement with water and with lime water were examined under microscope. Cement reacts with water very slowly first, but more vigorously after several hours. With lime water, reaction starts almost instantaneously and becomes slower afterwards. Authors come to conclusion that hydration of monocalcium aluminate with water is reversible and slowly, while reaction between monocalcium aluminate, calcium hydroxide and water is irreversible and takes place very rapidly, which fact causes quick setting of mixtures of high alumina cement with other calcareous hydraulic materials.—A. E. BEITLICH

The degree of separation by air separators. H. MADEL. *Zement* (Germany), Oct. 9, 1930, V. 19, No. 41, p. 958-63.—Directions for valuation of air separators, which were given by P. Rosin and E. Rammler in previous article (cf. *Zement* (Germany), 1929, V. 18, No. 26-32), are discussed. Three conditions must be fulfilled by efficient air separator: (1) separator must produce a material, which has most suitable composition for any desired purpose, (2) separator must concentrate fine flour, which is present in raw material, in final product, and (3) separator must prevent undesirable oversize particles from reaching final product. Efficiency is 100 per cent only when all three conditions are completely fulfilled. A number of equations are given for a mathematical expression of above facts which enable calculation of efficiency. Same conditions are shown also in graphs. Reply by Rosin and Rammler is given.—A. E. BEITLICH

Effect of gypsum on the decomposition of tri-calcium silicate by heat. THOMAS F. MULLAN *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 62.—Work of Myers is quoted which shows that calcium tri-silicate is decomposed by temperatures lower than that of the cement kiln. In addition it was found the decomposition to be accelerated by presence of gypsum. Experiments were made with three clinkers heated with amounts of gypsum varying from 2 per cent to 12 per cent. Free lime was determined after heating by method of Lerch and Bogue. Long heating with gypsum gave almost the free lime required by decomposition of tri-calcium silicate to di-calcium silicate and free lime. In some cases there was a slight unexplained excess of free lime. Tests of heating gypsum to 1000° C. indicated that none of the free lime was contributed by the gypsum.—EDMUND SHAW

Thielekiln. A. B. HELBIG. *Zement* (Germany), Aug. 7, 1930, V. 19, No. 32, p. 745-9.—In new type of high efficiency shaft kiln described, raw mix and powdered coal are stored in two bins under which two automatic scales are installed. Electric control device operates both scales and dumps material at same time. Screw and elevator deliver mixture to second screw in which materials are mixed and wetted with water. Wet material is pressed through steel plate with many holes about ½ in. in diameter by means of heavy steel rolls. After passing plate, material breaks up in small cylinders and is distributed over whole opening of kiln. Gases which pass upper layer have temperature of 140° to 180° C. They contain 32 per cent carbon dioxide and below 1 per cent carbon monoxide and oxygen. About 12 to 16 in. below surface begins sintering zone of clinker. Carbon dioxide of raw mixture forms with carbon of fuels carbon monoxide, which combines with oxygen of air for combustion to carbon dioxide. Combustion air is preheated. Total height of kiln is 25 ft., and its output is 120 to 140 tons of clinker per day.—A. E. BEITLICH

The Italian cement, lime and gypsum industries. R. DECKERT. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 76, p. 1229-30. The Italian manufacturers of cement, lime and gypsum are in a single association. They make alumina, high strength portland, standard portland and natural cements, the

last class making up about 55 per cent of total. There are 140 cement mills with 800 kilns and 20,000 employees. Firms making cement products number 2,440 with 21,746 employees. Lime, mostly hydraulic, is burned in 586 plants with 1000 kilns and 6,000 workers. Gypsum is burned in 115 plants by about 1000 workers. Annual production in millions of barrels (200 kg.) is about: natural cement 22, portland cement 17, lime 12.5 and gypsum 4.—F. O. ANDEREGG

Stirring and mixing of slurry. A. VOGT. *Zement* (Germany), Aug. 21, 1930, V. 19, No. 34, p. 809-10.—Equipment consists of cylindrical reinforced concrete tank, about 20 ft. wide and 33 ft. high, with conical bottom for mixing slurry in wet cement plants. Its capacity is about 6,700 cu. ft. Compressed air of 2 atm. pressure, which enters tank at bottom is distributed and stirs slurry. A second cylinder, open at top and bottom, diameter of which is about half of that of tank, with openings in its sides, is held in vertical position in center of tank. Upper edge of cylinder must always be below surface of slurry. Slurry is forced upwards through inner cylinder, flows over its top and through side openings and moves downwards between walls of cylinders. When it becomes necessary to add dry raw material to slurry, a cylindrical sieve will be installed between both cylinders. Material circulates 20 times per hour. Calculations show speed of slurry and power consumption.—A. E. BEITLICH

Use of powdered coal and crude oil in the rotary cement kiln. AFTON J. BLANK. *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 53.—Records are given from kilns using powdered coal and crude oil as fuel. Heat used varies from 1,212,840 B. t. u. to 1,745,264 B. t. u. Both kilns used crude oil, but with one exception the powdered coal fired kilns gave higher clinker outputs. Author concludes that wide differences in heat consumption are partly due to composition of cement but are mainly due to differences in kiln operators. Not only fuel economy but the life of the kiln lining depends upon the skill of the burner. Instances are cited where unskilled men have burned out expensive linings of high alumina brick in a short time.—EDMUND SHAW

Studies on calcium ferrites and iron cements II. SHOICHIRO NAGAI AND KATSUHIKO ASAOKA. *Kogyo Kwagaku Zasshi* (*J. Soc. of Chem. Ind.* (Japan), May, 1930, V. 33, Supplemental binding, No. 5, p. 161-4.—Pure dicalcium ferrite was prepared by extraction of free lime from product obtained by heating of mixtures of $3CaO:1Fe_2O_3$ and $5CaO:1Fe_2O_3$ at 1350° C. and 1550° C. It is stated that dicalcium ferrite is ferrite with highest lime content. Monocalcium ferrite $CaO.Fe_2O_3$ was prepared by heating a mixture of 1 CaO and 1 Fe_2O_3. Combination begins at 1000° C., proceeds very slowly up to 1150° C. and ends at 1200° C. when entire mass melts. Solubility of monocalcium ferrite in hydrochloric acids of different concentrations (0.5 to 3.0 normal) was determined. Method was derived to separate dicalcium ferrite and monocalcium ferrite from their mixtures. Method is based on the different solubilities of these compounds in hydrochloric acid.—A. E. BEITLICH

Comparison of balls and cylpebs as grinding media. K. KOYANAGI. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 58.—Although grinding capacity of balls is larger than that of cylpebs it is rapidly reduced by cement that sticks to the balls. On this account the balls show the worse result in grinding. Cylpebs probably grind less by impact and more by friction than balls so adhering cement is rubbed off. The author found more "flake" cement in ball than in cylpeb product which tended to confirm this theory. Opening a 14-ft. ball mill that had been stopped suddenly showed a rapid increase in fines up to 6 ft. after which the curve tended to flatten out. With cylpebs the relation between fineness and length was more regular, the curve being nearer to a straight line.—EDMUND SHAW

Tube mill grinding. A. B. HELBIG. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 870-3.—Author describes different degrees of fineness to which materials in cement industry must be ground. Raw materials must be so fine, that sintering in kiln takes place at lowest possible temperature. Raw mixture and burned material will gain in uniformity with increasing fineness. All particles should have approximately same size so that they reach necessary temperature in kiln at same time. Coal for shaft kilns should be ground finer than it is now in practice in order to accelerate sintering process. For rotary kilns, coal should be uniform in particle size. Its fineness depends on percentage of volatile material. Cement should be ground very fine to uniform sizes without making operation uneconomic. Tests were made to study grinding process in compartment mills and suggestions are given for economic dimensions and practical installations of air separators. (cf. Grosse, Foerderreuther and Rammler, *Zement* (Germany), Feb. 27 and March 6, 1930, V. 19, No. 9-10, p. 189-94, 214-7; *J. Am. Concrete Inst.*, May, 1930, V. 1, No. 7, Abstr. Sect. p. 92; Naske, *Tonindustrie Zeitung* (Germany), March 20, 1930, V. 54, No. 23, p. 386.)—A. E. BEITLICH

Researches on the rotary kiln in cement manufacturing. Part II. GEOFFRY MARTIN. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 60.—This part discusses methods of calculating the fuel consumed in a rotary kiln by analysis of coal, exit gases, raw mix and clinker. Three methods are given. The first is the author's own, based on the fact that fuel consumption is a function of volume of CO_2 per 100 volumes of air-free nitrogen in exit gases. Instructions are given for making a table or curve from which coal consumption may be read after CO_2 volume of exit gases has been found. Second method is a calculation made from volume percentage of nitrogen in exit gases. Third is by the Kuhl's formula which the author has found to give low results but which he thinks would give correct results with calibration of instruments and calculated constants.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part III. GEOFFRY MARTIN. *Rock Products*, Sept. 13, 1930, V. 33, No. 19, p. 57. This reviews the author's work for the British Portland Cement Association on heat losses due to formation of carbon monoxide and admission of excess air. Statistics from 2139 analyses of exit gases show best result is obtained with present conditions when exit gases show 1.1 per cent free oxygen. But in practice this figure is never maintained, the actual content flucterating from 0 per cent to 7 per cent. Author says this indicates that burners are vigilant in watching burning and constantly correcting air supply, but that the eye is not a sufficient guide. He proposes and explains the use of an indicator by which oxygen in the exit gases may be kept between 1 per cent and 2 per cent to save from 50 to 150 tons of coal a week in operation of a 200-ft. kiln, according to the degree of prevailing mismanagement. In many cases increases of 10 per cent to 40 per cent output may be obtained. A number of tables and charts are presented, with formulas needed for kiln testing and their derivation.—EDMUND SHAW

A development in air classification. GEORGE ADAMS ROLFE. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 47.—The air classifier described is a tower through which the material falls against an ascending current of air. Outlets with catch basins allow part of the air to escape at different heights carrying a sized product with it. The velocity of the air is greatest at the bottom and least at the top. Experiments have been made on a unit handling from three to five tons per hour. Although originally designed for separating mica, the classifier may be applied to the grinding of cement and is particularly adapted to slag grinding. The author points out that the device has no moving parts and there is nothing in it that can wear out.—EDMUND SHAW

Decrease of water content of slurry in manufacture of portland cement by wet process. PETER P. BUDNIKOFF, G. W. KUKOLEW AND W. M. LESCHOJEW. *Kolloid Z.* (Germany), Sept., 1930, V. 52, No. 3, p. 341-8.—Advantages and economy of wet process are characterized. New method is described, which enables reduction of water content of slurry without decreasing fluidity. Investigations were based on normal mixtures of limestone and marl with 48 per cent water. Heat calculations and viscosity determinations of this mixture were made. Additions of certain bases, like Na_2CO_3, Na_2SiO_3 or NaOH in concentrations from 0.005 to 0.15 normal, increase viscosity. Peptonization is caused by OH-ions. Sodium silicate shows greatest effect, while sodium hydroxide absolutely destroys fluid properties. Additional studies were made with molasses solutions. A theory is derived explaining coagulation and peptonization. Decrease of water offers many advantages, less heat is consumed, grinding process is accelerated, hardness of water is reduced by additions and dimensions of equipment can be reduced. (cf. *Zement* (Germany), Jan., 1930, V. 19, No. 5, p. 96.)—A. E. BEITLICH

New device for the removal of rings in rotary kilns. *Zement* (Germany), Oct. 9, 1930, V. 19, No. 41, p. 963-4.—A new type of cement gun is described. Special features of apparatus are great mobility, ease in aiming at any desired point and low expenses. Bullet weighs about 4 ounces.—A. E. BEITLICH

Researches on the rotary kiln in cement manufacture. GEOFFREY MARTIN. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 49—This is the first of a series of 27 articles. The author says that a complete theory of the rotary kiln has been worked out. Efficiency is barely 19 per cent showing a wide scope for improvement in design and operation. With a slurry containing 40 per cent water, fuel consumption should be about 21 tons standard coal for 100 (long) tons clinker produced. Of supreme importance in fuel economy is maintenance of as high a flame temperature as possible in combustion zone. Only B. T. U.'s above 805 C. can be directly utilized for forming clinker and merely preheat the slurry. Losses of heat by radiation and convention, both inside and outside the kiln are far more important than they were thought to be.—EDMUND SHAW

Some errors in rapid methods of mix control. S. L. MEYERS. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 60.—Reasons why the acid-alkali titration, ignition loss and lime soluble in HCl from raw or calcined raw mix are not absolute criterions of what will be the lime in burned clinker, are acid-insoluble compounds in the raw mix and variations in composition of the dust carried up the stacks of kilns by the waste heat gases. A long list of acid-insoluble lime minerals is given, the commonest those of the pyroxene group, such as augite and hornblend, and some of the feldspars and garnets. There are also lime minerals only partly soluble in acid. Dust losses vary with materials in raw mix. With high and low limestones the high grade limestone powders readily after calcination and much goes out with the stack dust, thus lowering the lime in the clinker. The effect is apt to be reversed where clay is used as the low, as the clay is more easily picked up by the exit gases.—EDMUND SHAW

Influence of use of high quality aggregates on quality and costs of concrete. "Einfluss der Verwendung von Edelzuschlag auf die Guete und die Kosten von Beton." PAUL HERTEL. 1930, Charlottenburg 2 (Germany), R. M. 2.00. Reviewed in *Zement* (Germany), Sept. 18, 1930, V. 19, No. 38, p. 906.—Results of comprehensive investigation are published concerning effect of aggregate gradation on consistency, weight, strength and water permeability of concrete. Costs of aggregates and concretes are compared. Tables and graphs explain this investigation.—A. E. BEITLICH

Rapid determination of magnesia in portland cement and raw materials. AUTHUR J. POOL. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 80.—The method of determining magnesia by weighing as $Mg_2 P_2 O_7$ is accurate but tedious. The author prefers a volumetric method. The sample is dissolved in dilute hydrochloric and nitric acids and iron and alumina are precipitated with ammonium hydroxide. Lime is precipitated as calcium oxalate and determined by potassium permanganate titration. Magnesia is determined on the filtrate from lime by precipitation with known volume of sodium hydroxide and titrating the filtrate with hydrochloric acid for excess. Article notes several details which must be carefully attended to for a correct result. It is especially important that all the ammonia be expelled before titrating. Magnesia is calculated from the amount of sodium hydroxide required to precipitate it.—EDMUND SHAW

Small-piece testing method for strength of cement mortars IV. S. NAGAI. *Kogyo Kwagaku Zassi* (*J. Soc. of Chem. Ind.*) (Japan), March, 1930, V. 33, Supplemental binding, No. 3, p. 290-5.—New method was derived and investigated by author to test strength of cement mortars with test pieces, which are much smaller than normal sizes. Tests were made with portland cement, blast furnace slag cement and several high alumina cements and one Soliditit cement comparing obtained results with normal testing methods. It was found, that in general lower results were obtained after 28 days combination storage than after 28 days water storage, when tested by small-piece testing method. No increase in strength of high alumina cements was observed after long water storage. Author claims, that this fact is due to solution of a tricalcium aluminate which is formed during hydration of $5CaO.3Al_2O_3$ and monocalcium aluminate.—A. E. BEITLICH

Are you getting the true lime value on your raw mix analysis? ROBERT R. KING. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 60.—Where lime is determined by either gravametric or volumetric methods after fusion with alkaline carbonates an error as much as 2 per cent to 3 per cent may be introduced. Tables show that fusion with varying amounts of $Na_2 CO_3$ from 0.2 gram to 8.09 gave from 75.48 per cent to 80.33 per cent apparent $Ca CO_3$ content. This is because sodium and potassium salts formed in fusion cannot be washed out in spite of their high solubility. Remedy suggested is to use as little alkaline salt for fusion as possible. The author states that with 20 per cent insoluble in the 0.5 grams sample of raw mix only 0.1 grams of fusion mixture is necessary.—EDMUND SHAW

Raw grinding and slurry control. R. J. BINFORD. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 41.—Maximum production in a grinding plant is only accomplished by having proper size of grinding media in each mill compartment. The author gives the formula for the proper size of largest ball. Largest practical size is 5-in. dia. as with larger balls repairs on tube mill are heavy. The feed should therefore all pass a 1½-in. screen and nearly all should pass a 1-in. screen. Control of slurry covers: Grinding to the required fineness for burning and for desired quality, maintaining as low moisture content as will allow the slurry to be handled easily and quickly and maintaining a correct chemical balance of different compounds necessary in finished cement. The finer the grinding the better the results in the kiln but as fine grinding is expensive materials are ground to point where maximum demand for quality is satisfied. For economy's sake water in slurry is kept as low as is consistent with quality. At the West Penn. plant, where the author has charge of mechanical departments, economical point is 33 per cent to 35 per cent moisture.—EDMUND SHAW

Standardizing cement sieves. E. TAYLOR. *Rock Products*, June 21, 1930, V. 33, No. 13, p. 85.—The author gives his method, used by the Cebu Portland

Cement Co., Naga, Cebu, P. I., for correcting the fineness sieve test. A 200-mesh sieve is checked and calibrated by the U. S. Bureau of Standards. A large bottle of cement, fineness of which has been determined by this sieve, is kept in laboratory and each tester is required to test a sample of this cement with sieves actually used in testing the shift's run. Results are entered on a card without correction. Results are averaged by laboratory chief and compared with true fineness of the bottle cement and the necessary corrections determined for sieves in use. These corrections are applied until new corrections are ordered. System insures correct results and has the advantage that checked and standardized sieve is used only occasionally.—EDMUND SHAW

Internationalization of standards. V. M. ANZLOVAR. *Zement* (Germany), Oct. 2, 1930, V. 19, No. 40, p. 942-3.—Standard specifications for cements were revised in several countries. Holland and Italy were first to work out specifications for high alumina cements, Spain followed their example. High alumina cements must contain more than 40 per cent Al_2O_3 and less than 12 per cent Fe_2O_3; fineness must be: not more than 0.5 per cent residue on 75 mesh sieve and not more than 6 per cent residue on 175 mesh sieve; time of initial set shall not be less than 30 minutes and final set shall not be later than 4 hours. Tensile strength is tested with neat cement briquets. New concrete testing method for compressive strength is introduced. Standard 7.9 in. cubes are prepared from following mixture: 659.2 lbs. cement per 35.3 cu. ft. concrete, 30 cu. ft. broken stones (0.37 to 0.59 in.), 14.1 cu. ft. sand (0.04 to 0.06 in.) and 34.3 gallons of water per 35.3 cu. ft. concrete. Compressive strength after 1 day must be at least 3150 lb. per sq. in. and after 28 days 3270 lb. per sq. in. Similar specifications are given for high early strength cements. (cf. Platzmann, *Zement* (Germany), Jan. 2 and 16, 1930, V. 19, No. 1 and 3, p. 4-7, 49-52; *Ibid*, July 24, 1930, V. 19, No. 30, p. 701-2; *J. of the Amer. Concr. Inst.*, March, 1930, V. 1, No. 5, Abstr. Sect. p. 48; *Ibid*, Oct., 1930, V. 2, No. 2, Abstr. Sect. p. 23.)—A. E. BEITLICH

German standard specifications for portland cement, iron portland cement and blast furnace cement. *Zement* (Germany), Sept. 18 and 25, 1930, V. 19, No. 38-39, p. 890-9, 915-9.—New proposed standard specifications, open for objections until Nov. 1, 1930, differ from old specifications in following points: Specifications for high testing cements are added, limits for compressive and tensile strength are raised considerably, definition for blast furnace slag cement was changed. Specifications are divided into three parts. Part 1. Definition and properties. (1) Portland cement: loss on ignition should be less than 5 per cent, MgO less than 5 per cent and SO_3 less than 2.5 per cent (ignited base). Ratio between CaO and sum of soluble SiO_2, Al_2O_3, Fe_2O_3 and MnO should not be smaller than 1.7. Additions of foreign materials after burning process must be below 3 per cent. Not more than 35 per cent of raw materials, dried at 212° F., should be retained on 175 mesh sieve. (2) Iron portland cement: must contain at least 70 per cent portland cement and not more than 30 per cent granulated basic blast furnace slag. Ratio between 1 CaO + 1 MgO + $\frac{1}{3}$ Al_2O_3 and SiO_2 + $\frac{2}{3}$ Al_2O_3 in slag should be greater than 1. (3) Blast furnace slag cement: this cement is ground mixture of granulated blast furnace slag (having same ratio as for iron portland cement) and 15 to 69 per cent portland cement. Slag should not contain more than 5 per cent MnO. CaO content is in general below 55 per cent. Fineness for all three cements must be 2 per cent retained on 75 mesh sieve and not more than 25 per cent retained on 175 mesh sieve. Hardening should not start sooner than 1 hour after mixing cement and water. Following strength data must be reached of 1:3, cement: standard sand, mixture:

	Cement		High Testing Cement	
	Compr.	Tens.	Compr.	Tens. Str.
After 3 days water storage	....	...	3555	355.5
After 7 days water storage	2560	256	....	....
After 28 days water storage	3910	...	....	....
After 28 days combination storage	4977	427	7110	569

Water storage means: 24 hours in moist closet then in water (63 to 68° F.). Combination storage means: 24 hours in moist closet, then 6 days in water and then 21 days in air (63 to 68° F.). Part 2 discusses sampling, preliminary tests, preparation of materials for standard tests, determination of fineness, initial and final set and soundness, also mixing, making, storage and breaking of cubes and briquets. Specimens for compressive strength tests are 2¾-in. cubes and for tensile strength are briquets with smallest cross section of 0.77 sq. in. Part 3 deals with standard sand, its chemical composition and grain sizes. Standard specifications for sieves are given. Apparatus for determining setting time, for mixing mortars, hammer apparatus and tensile strength tester are described. Exact dimensions of test pieces and their molds are given and also allowable tolerances for manufacture and wear.—A. E. BEITLICH

California's largest cement mill keeps up-to-date *Rock Products*, June 21, 1930, V. 33, No. 13, p. 41.—Oil fuel for the Santa Cruz Portland Cement Co.'s plant at Davenport, Calif., is pumped from tankers to storage tanks on shore about three miles from the plant. The line rests on the ocean bottom for one-half mile. This is one of the few places where oil is pumped from open sea to plant storage. Limestone is mined from glory holes above bulldozing chambers connected by chutes to two main haulage tunnels. Clay overburden supplies most of clay needed for the mix. Mining is safer than quarrying. Costs of opening one glory hole with bulldozing chamber and tunnels was $26,000, about 86¢ per ton for development. Blending begins by keeping coarse and fine limestone separate and combining as needed. The fines carry much clay. A second blend is made by drawing proportionately from "hi-low" bins containing crushed product. This blend goes to 40 steel hoppers and a mix from these goes to Hardinge raw grind mills with air separators, either Hardinge or Sturtevant. A heavy circulating load is carried. Grinding is 90 per cent minus 200-mesh. Hardinge mills finish grind clinker after it has been crushed in rolls in closed circuit with vibrating screens. Gypsum is calcined in Raymond "calcine-while-you-grind" mills. Six ball and six tube mills grind "oil well" cement which is ground finer and harder burned and approaches high early strength cement in its properties.—EDMUND SHAW

Modern cement testing laboratory. *Rock Products*, June 7, 1930, V. 33, No. 12, p. 75.—The new laboratory of the Southwestern Portland Cement Co., at El Paso, Texas, has been successfully designed to eliminate unnecessary noise. All grinding machines have texrope drives and are mounted on lead cushions. The rotap screening machine is placed in a cabinet lined with celotex and mounted on a lead cushion. The moist room is kept within 1° F. of 70°. It has double concrete walls insulated with tarred cork-board. There is one large door for entrance and two small doors for attending to specimens while they are setting up. A 5-h. p. Kelvinator and four Chromolox strip heaters control the temperature. They are regulated by two thermostatic mercoid switches. Sprays keep the air at almost 100 per cent humidity. A wet and dry bulb recording thermometer keeps a continuous record of temperature and relative humidity. Average range of temperature for 24 hours for practically all records is within 0.2°. Physical testing bench tops are of ¼-in. non-rusting steel plate. The bench for titrations has a glass plate with an electric light below.—EDMUND SHAW

Plant of Republic Portland Cement Co. in second year of production. *Pit Quarry*, Aug. 13, 1930, V. 20, No. 10, p. 17-34.—Wet-process plant with rated capacity of 3,600 bbl. per day has actually produced as high as 3,900 bbl. Raw material is quarried from argillaceous limestone or chalk which varies from soft earthy to fully solidified rock. A large bed of natural cement rock constitutes about 80 per cent of deposit. Broken stone from quarry is loaded by electric shovel into 8-cu. yd., side-dump, standard-gauge steel cars and hauled to plant by gasoline locomotive. Crusher will take feed as large as

2-cu. yd. and reduce it to ¾-in. in a single operation. One building houses the raw and finish grinding mills and the power equipment. The two types of limestone are fed, from the bins to two compeb mills. Slurry is ground to a fineness of 90 to 95 per cent through a 200-mesh sieve and is discharged into a horizontal sump and stirred by a double-ribbon conveyor. With a water content of 42 per cent it is handled by two 6-in. pumps, through a 10-in. line in a tunnel underneath the storage, to slurry tanks on opposite side. Same tunnel is used by pedestrians and for air, water, and electric lines. The six blending tanks with four correction tanks hold slurry for 48-hour run. Each tank has a mechanical and air agitator driven by motors through speed reducers. The chemist samples slurry in each blending tank while filling, and corrections are made by transfers. Two kilns are fired with natural gas at 1½ lb. pressure, and equipped with temperature recorders. Slurry enters the kilns through ferris-wheel feeders. Feed end of each kiln is within a dust chamber which connects by steel breeching with its 200-ft. reinforced-concrete stack. Clinker from kilns passes to coolers which discharge to the clinker storage pit. The clinker is taken by crane to finish-mill feed bins. Table feeders pass clinker and gypsum to compeb mills. A dust-collecting system on a mezzanine floor serves also as a ventilator. Pneumatic pumps in basement propel finished cement to tops of storage silos having total capacity of 153,000 bbl. Packhouse is a 4-story building adjacent to the cement storage. Cross screw extends outside to a loading track for loading bulk shipments through a 6-in. rubber hose. An air-jet in the hose keeps the line clear.—A. J. HOSKIN

Reactions during setting and hardening process of cement. (See PROPERTIES OF CONCRETE.)

Temperature and cement. (See PROPERTIES OF CONCRETE.)

PROPERTIES OF CONCRETE

Reactions during setting and hardening process of cement. TUTOMU MAEDA. *Zement* (Germany), Sept. 4, 1930, V. 19, No. 36, p. 842-3.—A theory is derived illustrating reactions which take place during setting and hardening periods and explaining hardening properties of cement. Certain constituents of cement form when mixed with water unstable compounds in a system containing one liquid phase. Second step is formation of oversaturated solution of stable compound. When a new solid phase precipitates from this oversaturated solution, it absorbs certain constituents of the liquid phase, which form a film around particles. Conditions for a proper hardening are: (1) particles, which separate from liquid phase, must be small, which gives great surface, (2) particles must be separated by adsorbed film, thickness of which lies in molecular range. No liquid phase should be present. (3) Attractions between the different particles and particles and film must be great, and (4) particles must have great strength. A number of experiments with ultra-microscope, which seem to prove existence of colloidal parts, are described. (cf. *Supplement to Scientific Papers of Inst. of Phys. and Chem. Research* (Japan) Tokyo 1928, V. 8.)—A. E. BEITLICH

Temperature and cement. OSCAR FABER. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 566-569.—Curves are given showing relation between temperature and time of setting for different kinds of cement. A case is cited where a recorded rise in temperature of 50 degrees occurred in three days after placing of the cement. Use of this property of heat production can be made by insulating concrete thus preventing concrete from cooling too rapidly until it is hard. Reference is made to

Bulletin No. 81, Engineering Experiment Station, University of Illinois, to point out the increase in the strength of concrete with increase in the temperature at which it is cured.—JOSEPH MARIN

Chemical characteristics of cement pipe lining. E. L. CHAPPELL. *Ind. Eng. Chem.*, Nov., 1930, V. 22, No. 11, p. 1203-6.—A brief review of literature regarding use of cement pipe lining and theories advanced to explain its effectiveness as a protective layer are given. Data are presented showing change in chemical composition that took place in several types of cement during exposure. The ages of the linings examined varied from one to two years with hot water exposure and from five to 60 years with cold water exposure. It was found that calcium is gradually leached out and iron hydroxide deposited and that this exchange did not result in disintegration of lining nor alter the effectiveness of the coating in preventing corrosion. The deposition of iron hydroxide will take place in a layer of sand and finally cement the sand into a hard coating. Cement pipe linings have been found to be excellent protection in tuberculating water and also good results were obtained with salt water and sulfur water. If CO_2 content in steam is high in steam return lines, it has a solvent action on both the iron and calcium compounds and destroys some cement linings.—ROY N. YOUNG

The elasticity of concrete. H. H. LANGDON. *Monthly Bull. No. 30*, Washington State College, April, 1930, V. 12, No. 5, 48 p.—Data were taken from three sources: student work, control specimens from campus construction jobs, and from field jobs. Special apparatus used to measure deformations, consists of two steel rings, each clamped to the specimen by three equally spaced screws. Connected between the rings are three equally spaced metallic bellows. The three bellows are interconnected by means of small tubes. Another small tube extends from one of the bellows to a glass tube placed at a small angle from the horizontal. Any deformation in the specimen forces a small quantity of water through the bellows and tubes into glass gage where it is easily read. A comparison was made between the results obtained with this type gage and with mirror type. Satisfactory agreement was found. A screw machine and hydraulic machine were used to produce loads. Two hundred and forty specimens were tested. Tests were conducted to prove that slight curve at lower end of stress-deformation diagram was due to a lack of accuracy in the weighing of loads, or in part due to the compressometer or to temperature changes, and is not indicative of true deformation within the concrete. The major conclusions state that: concrete is elastic in the same sense that steel is considered elastic; elastic limit occurs at approximately 37 per cent of ultimate; the relationship between elastic limit and ultimate stress is fairly constant; if elastic limit is not exceeded, concrete may be reloaded and same stress-deformation relations will be obtained; modulus of elasticity as found is considerably higher than that used in present day design; there is a relationship between ultimate strength and modulus of elasticity. There are also enumerated several conclusions that need additional verification. A resume of studies made by other experimentors in the same field is also given.—IRA L. COLLIER

Corrosion of structural steel and steel reinforcing rods encased in concrete. E. A. CROSS. *Engineering* (England), July 4, 1930, V. 130, No. 3364, p. 30.—Structural steel encased in concrete and steel rods in a reinforced concrete slab supported by the structural steel beams were noted in a 26-year old engine room of a Toronto store. The construction consisted of 18-in. steel beams, span 25 ft., spaced 3 ft. 9 in. on centers embedded in concrete which supported a 6-in. concrete slab with ½-in. diameter bars, 6 in. on centers, placed in the lower part of the slab. This floor supported an ammonia refrigeration brine tank which had been leaking for a long period. A pump room immediately below this floor contained steam mains which leaked and resulted

in condensation which kept the concrete near openings wet and at a temperature of near 100° F. Cracks were found running longitudinally with steel beams and the slab reinforcing rods. It was concluded that corrosion had been caused by the following factors: (a) Cracks in concrete slab due to omission of negative reinforcement over steel beams, (b) presence of water and brine solution on top of slab finding its way through these cracks, (c) excessive condensation at under side of slab at openings in floor combined with high temperature. In repairing, steel beams were left in position but their assistance was ignored and new concrete beams as well as new slab were poured around and above them. Mixture used was 1:2:3, giving strengths of 1500 and 1850 lb. sq. in. at 7 days. —G. M. WILLIAMS

Lightweight concretes. *Builder* (England), Sept. 12, 1930, V. 139, No. 4571, p. 436.—With modern framed structures the weight of filling walls needs be reduced to a minimum to permit use of lighter steel sections. Clinker concrete slabs are very light and fire-resistant, so long as no combustible matter is present in the clinker. Peeling of plaster from clinker concrete slab walls may be due to expansion of combustible matter in concrete. Clinker concrete should never be used to cover steel nor be reinforced, on account of its porosity and probable presence of sulphur in clinker. Slabs may be covered with plaster, or, if made with ⅜-in. crushed clinker and "fines," a smooth face may be obtained. Clinker concrete walls offer considerable resistance to penetration of moisture, notwithstanding their great permeability. Pumice concrete is another lightweight concrete, suitable for building, where its high cost is not prohibitive. While having the same advantages as clinker concrete, pumice concrete has no dangers from sulphur presence. The usual mix is six parts of pumice to one of cement, fines often being omitted, to give greater voids. Voids may be artificially produced by mixing a chemical with cement, causing it to bubble and become honeycombed with air pockets. Finely ground aluminium and zinc used in these patented processes, react with lime to form hydrogen. Considerable insulating properties against heat and sound are produced in these "cellular" concretes. Sawdust concrete is a further type of lightweight concrete, the inflammable nature of the aggregate and considerable amount of its expansion, when wet, being most serious difficulties to overcome. Addition of calcium chloride to gauging water may obviate these difficulties, producing a "half-weight" concrete. By mixing small particles of ice with cement and sand, and moving the molds containing mix to a warm room, light cellular concrete may be obtained. The ice melts and hydrates the cement which forms a thin wall round the spaces occupied by the ice. Column shuttering may be dispensed with if hollow precast blocks of lightweight concrete are placed one on the other and filled with concrete and reinforcing steel. Fire resistance is thus considerably increased.—JOHN E. ADAMS

Deterioration of structures in sea water. *Can. Eng.*, Oct. 28, 1930, V. 59, No. 18, p. 593.—Tenth interim report of Institute of Civil Engineers, Great Britain, refers to experiments on the measure of resistance of reinforced concrete to deterioration by sea air and sea water. Three hundred reinforced concrete test pieces are being prepared for exposure in artificial sea water, in a sea water tank at Sheerness Dockyard and in the sea at Takoradi on the Gold Coast. The variables include 4 different kinds of cement with trass replacing a part of the cement or part of the aggregate. Three proportions are used and the medium richness will include wet and dry consistencies. Reinforced concrete test blocks were examined at Brisbane after 11 years exposure. Tar coatings have retarded but have not prevented deterioration of embedded steel.—G. M. WILLIAMS

High-early-strength concrete. S. RORDAM. *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 45.—High-early-strength concrete may be made by using special quick-hardening cements, or from ordinary portland cement by

control of the water-cement ratio. The author believes that latter method will give desired strength in specified time for least money and that it offers better insurance against unexpected stresses at later periods. Results of a comparative experiment made under uniform conditions are given in tables and diagrams. Water-cement ratio of portland cement concrete was controlled to give same strength as that of high-early strength concrete at three days. Considerably higher strengths at 28 days and 90 days than that of the high-early-strength concrete were obtained.—EDMUND SHAW

Control of the manufacture of concrete. LUIGI SANTARELLO. *L'Industria Italiana Del Cemento* (Italy), Feb., 1930, p. 16-19.—Italian concrete industry fails to apply fully available information in regard to concrete quality control as a good deal is left to the judgment of the laborer at the mixer. Quality of concrete depends on quality of materials and how they are proportioned. Cement is accepted as of good strength. It is suggested that aggregates be subjected to petrographical analysis to detect decomposable material. Feldspar is cited as such an objectionable material not indicated by usual analysis. Combination of aggregates on basis of laws of Fuller or Bolomey is recommended to give economy and density. Water-cement ratio is stressed as the one most important factor to control. Proper apparatus is available for measuring materials for concrete, and should be used.—C. G. CLAIR AND M. N. CLAIR

Exposed surfaces of concrete and their protection. A. B. MACMILLAN. *J. Boston Soc. C. E.*, Oct., 1930, p. 467-473.—The causes of the disintegration of exposed concrete surfaces are discussed and water is indicated as the primary factor. Water weakens concrete through solution of hydrated cement compounds, expansion or freezing and the corrosion of the steel. It gains access to the concrete mass through cracks and porosity of the concrete. The precautions necessary to delay disintegration are to use proper materials, proportions, mixings, placing and curing procedure in the original construction. In addition integral water proofing material may be applied but author favors sealing concrete surface with an elastic varnish and covering varnish by a paint. Detail specifications and procedure for repairing the surface of a reinforced concrete building are given. All rusted reinforcing steel was cleaned and painted with asphalt paint. Loose concrete was removed and replaced by 1:2 mortar mixed five hours before use. The surfaces were painted with two coats of concrete paint. The total cost was $6,048 or 12¢ per sq. ft. of gross surface area. This gives a cost of 1¢ per sq. ft. per year cost to owners for surface deterioration.—MILES N. CLAIR

Ready mixed concrete. MILES N. CLAIR. *J. Boston Soc. C. E.*, Oct., 1930, p. 474-480.—The lack of durability of concrete structures in the past is due to failure to appreciate value of definite control of the proportioning, placing and curing of the concrete. Control of concrete quality is too complex a matter to be left in hands of usual concrete foreman, and an engineer of special training is required. Small projects cannot stand expense of such control. Ready mixed concrete operations are large enough to warrant expense for control of quality. The several systems for producing ready mixed concrete and equipment used are described and their limitations discussed.—MILES N. CLAIR

Reinforced concrete chimney stacks. *Builder*, Oct. 17, 1930, V. 139, No. 4576, p. 664.—In reply to request for information concerning suitable aggregate for a reinforced concrete chimney stack, 150 ft. high, 7 ft. 3 in. to 9 ft. 9 in. diameter internally, with temperature of flue gases at base 750° F. and a firebrick lining to be carried up 25 ft., Building Research Station states that deterioration would be due to effect of heat on aggregate and effect of gases on concrete and cement, and stresses due to temperature gradient through the walls. Ordinary clean gravel or crushed stone should withstand the heat since

the temperature would be less than that at which calcium carbonate decomposes. Portland cement, however, would commence to dehydrate slowly and lose strength mainly on inner surface, near base. Effect of sulphur gases in fumes would only cause surface reactions, provided the concrete is dense and well compacted. Stresses due to unequal temperature and drying conditions would probably cause shrinkage cracks.—JOHN E. ADAMS

Water permeability of cement pipes. E. SUENSON. *First International Congress for Conc. Rein. Conc.*, Liege (Belgium), Sept. 1930; Publication No. VII-2, 19 pp. Editor: La Technique des Travaux, Liege.—Test pipes 3½ in. long, with 2 in. outside diameter, and walls ⅜ in. thick were made of ordinary portland cement and sand of 0.2 in. particle size in mixtures ranging from 1:2 to 1:5. Corresponding water additions were 9.2 to 8.0 per cent. Pipes remained in moist closet for 24 hours, then half of samples were stored in air saturated with water, the other half under water for 27 days. The 28 day old test pipes were then kept in air until used for tests. Volume of pipes was determined by immersion in mercury and weighing overflowing mercury. It was found that specific gravity decreases with decreasing cement content. Specific gravity is greater after air storage than after water storage. Experiments to determine water permeability were made with simple apparatus with pressures ranging up to 8 ft. water pressure, kept constant throughout tests. Theory concerning flow of water through concrete was derived. Formulas and graphs show changes of flow with increasing pressure, relation between velocity of flow and internal pressure in voids and velocity and pressure in cylindric walls. Amount of water in grams which passed walls of pipes per minute was determined under different pressures and conditions. Permeability is lowered considerably with increasing cement content. Impermeability of pipes is several hundred per cent greater when specimens are stored in water than in air storage. Storage conditions are of much greater importance than composition of mix. Lean mixtures can be made dense by storage under water. Admixtures like water-glass or soap solutions have only minor influence on water impermeability. When water is slightly acid cement will be dissolved, while ordinary water and distilled water make concrete more dense. This is partly due to swelling of cement particles and partly to infiltration of impurities. Bacteria in water have also improving effect on concrete pipes even when they are made from lean mixtures such as 1:10.—A. E. BEITLICH

ENGINEERING DESIGN

BRIDGES

Test of a reinforced concrete bridge. M. LANOS. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation in *Concrete Constr. Engr.* (England), Oct., 1930, V. 25, No. 10, p. 569-571.—Tests were made to determine effect of impact on a bridge carrying Rue Lafayette over lines of the Compagnie de l'Est in Paris. Deflections were measured for various members of the bridge. Readings were taken with loads passing over bridge at different speeds. The general conclusion drawn from the tests showed that, in bridges of this type, speed and shock do not sensibly increase stresses in principal beams. On the other hand, they produce stresses greater than those calculated in the cross girders, stringers, and the slab below the roadway.—JOSEPH MARIN

Design of road bridges. C. S. CHETTOE. *Rds. Rd. Constr.* (England), Oct. 1, 1930, V. 8, No. 94, p. 332.—Surface finish: If concrete bridges are faced with brick or masonry, dowels are not necessary, provided facing is suitably toothed into concrete backing. In exposed concrete face work, finish may be produced by use of colored cement, or special aggregates. Such facing is usually

only a few inches thick, and separated from the main body of concrete by a metal shutter, lifted as concreting operations proceed. Both kinds of concrete set together and become monolithic. Use of colored cement blocks avoids risk of patchiness. When special aggregate is used, it is usually crushed local stone. The surface skin of cement has to be removed, by some suitable method such as bush hammering or scrubbing with a wire brush or use of chemicals. The molds may also be coated with special retarding solutions, so that the surface easily brushes off with scrubbing. Further effects may be obtained by polishing with carborundum, or painting with waterproof paint.—JOHN E. ADAMS

Reinforced concrete canal bridges in Belgium. M. BOUCAU. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation in *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 561-563.—To conform with clearance necessary for the passage of barges and with topography of the land, the depth of construction available was only 2 ft. 10 in. in one case. A concrete arch bridge would not permit sufficient clearance and a steel bridge would be too costly. A satisfactory solution was found by constructing a portal with hinges at low level.—JOSEPH MARIN

BUILDINGS

High buildings of steel or concrete? K. SCHAECHTERIE. *Teknisk Tidskrift* (Sweden), Sept., 1930, No. 9, p. 124-126.—An office-warehouse building, five stories high and with a live load of 51½ and 206 lb. per sq. ft. for the office and the warehouse respectively was figured for cost and a comparison made between cost of a concrete and of a steel structure. The concrete building was found to have 1 per cent less effective floor area and to weigh 25 per cent more than the steel building. The cost for the former was 3.25 Mark against 3.73 Mark per sq. ft. effective floor area for the steel building, a difference of 15 per cent.—O. Albert

Design of wind bracing. EDWARD SMULSKI. *J. Boston Society Civil Eng.*, Nov., 1930, p. 491-523.—Details of the design of wind bracing are presented for tall buildings both of reinforced concrete and of steel, with particular attention to design of the connections of the wind bracing for steel structures. Reinforcement is required in concrete wind braces at the column to resist both positive and negative bending moments, the zone of negative bending moment extends further from the column than for an ordinary restrained beam and diagonal tension reinforcement may be required for the entire length of the beam to take care of the shear.—MILES N. CLAIR

DAMS

Unique cutoff construction and arched foundation features of Rodriguez dam. (See FIELD CONSTRUCTION—Dams.)

MISCELLANEOUS

Construction of quay at Rotterdam. W. F. VAN DIJCK. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation, *Concrete Const. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 557-61.—The quay walls will be 8500 ft. long and are designed to allow for an ultimate depth of water of 39 ft. Vertical walls were adopted to permit simplification in placing concrete, and though greater amounts of steel and concrete will be required than by using battered walls, the ultimate unit cost will be less. Details are given of method of construction. The layout of the yard and method of constructing caissons are points of interest. Coulomb's

method of calculating earth pressure was adopted, using values of $\phi = 30$ degrees and $\phi = 40$ degrees for sand below and above water level respectively. —JOSEPH MARIN

New railway roadbed construction of reinforced concrete. *Zement* (Germany), Sept. 18, 1930, V. 19, No. 38, p. 904-5.—A new type of foundation for railroad tracks is described, which can be used with great advantage for railroad bridges where vibrations of structure must be reduced to minimum. Tracks are connected to concrete foundations by means of heavy steel springs. Experiments undertaken on main line of Austrian railroad gave excellent results. (cf. Wirth, *Organ f. d. Fortschritte d. Eisenbahnwesens* (Austria), Nov. 1929, No. 21.)—A. E. BEITLICH

Concrete retaining walls. E. W. ANDREWS. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liège, 1930. Abstracted from *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 572-573.—Cross sections and relative economies of plain and reinforced concrete walls designed to fulfill as nearly as possible the same conditions for three different forms of construction and two different theories of earth pressure are discussed. The three forms of construction are: (1) mass concrete, (2) "slightly" reinforced concrete, and (3) "conventional" reinforced concrete.—JOSEPH MARIN

Examples for calculation of reinforced concrete structures. "Exemples de calculs de constructions en beton arme." LEON COSYN. Editor: Librairie Politechnique Ch. Béranger, Paris and Liège (Belgium), 1928, 3rd edition, Fr. 60.00. Reviewed in *Zement* (Germany), Sept. 4, 1930, V. 19, No. 36, p. 858.—Book is continuation of: "Practical treatise of reinforced concrete structures" (Traité pratique des constructions en béton armé) by same author. Fundamental principles of calculations, which arise in all fields of reinforced concrete construction are discussed and explained with numerous examples. In its seven parts, book deals with scaffolds; prismatic structural parts exposed to compression, tension, flexure or torsion; discussion of total structures, and gives tables for use by designer and contractor.—A. E. BEITLICH

Parabolic sheds in reinforced concrete. L. BAES AND J. VERDEYEN. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liège, 1930. Abstracted from translation in *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 563-565.—These sheds are used for storage of chemicals of a highly cohesive nature. Products of this kind cannot be stored in ordinary silos. Experience has proved that best cross section is parabolic or nearly so. A three hinged arch was used. Economical ratios between rise and span are given and dimensions and details of construction are outlined.—JOSEPH MARIN

Shaping of incomplete reinforced concrete hinges. ALEX JESINGHAUS AND OTTO BIELIGK. *Zement* (Germany), Sept. 4 and 11, 1930, V. 19, No. 36-37, p. 850-5, 873-9.—Critical study was made investigating influence of shaping of joints and placing of steel in reinforced concrete hinges. Mathematical calculations illustrate occurance of stresses in hinges of different shapes and with different reinforcements. Certain types are discussed and their application and advantages characterized. Great care must be taken in placing steel reinforcements, number of which should not exceed the necessary limit, so that uniform concreting can be done.—A. E. BEITLICH

Oregon highway bridge to be built by Freyssinet arch method. *Eng. News Record*, Aug. 21, 1930, V. 105, No. 8, p. 290.—Utilization of Freyssinet system of concrete arch construction will be feature of major interest in building Rogue River highway bridge now under construction for Oregon State Highway Commission. Seven pairs of arch ribs each having 222-ft. span and

rise of 47 ft. make up main portion of structure. At either end will be 144 ft. of concrete viaduct approach composed of 16-ft. arches, making a total length of 1,938 ft. Abutments at ends of main arch system will rest on rock and those for six intermediate piers, including two abutment piers, will rest on foundations of piles driven in gravel. Intermediate abutment piers divide arch system into a three-span group and two two-span groups on either side. The piers are designed as yielding or elastic, believed to be an innovation in use of Freyssinet method. In this method rib-shortening and shrinkage stresses are reduced by leaving an opening at crown of each rib in the concrete and inserting hydraulic jacks to allow for adjustments. These jacks permit the position of the arch to be maintained or corrected, following the construction period, until sufficient time has elapsed for rib-shortening to take place. Because of unusual design, the structure is to be a cooperative research project with U. S. Bureau of Public Roads and State of Oregon as participants.—D. E. LARSON

Design of road bridges. C. S. CHETTOE. *Rds. and Rd. Constr.* (England), August 1, 1930, V. 8, No. 92, p. 257; No. 93, p. 297.—Retaining walls: Mass concrete walls, rarely employed for greater heights than 10 ft., should be battered slightly to care for settlement under the toe. Rankine's Theory is still mainly used, and is applied to stability against sliding and overturning. Respective factors of safety should be 1.5 and 2.0. For live load, a 3 ft. surcharge is usually satisfactory to cover effect of Ministry of Transport train. All walls should be backed with dry fill, and provided with weepholes. It is essential that concrete in the toe of wall be formed against virgin soil, without fill, and that no open drain be constructed in front of wall. Otherwise passive pressure will be destroyed and sliding result. For reinforced concrete walls up to 12 ft. high, L-walls without counterforts, consisting of vertical slab, toe and heel are common. The resistance to sliding may be increased by sloping the footing slab. If the toe is much greater than the heel, to avoid excessive cutting into the bank, the stability factor is lessened, but by making the footing slab butt against the road slab, good resistance against sliding is provided. For heights greater than 10 to 12 ft. counterforts with horizontally spanning slabs are more usual. Positive moments are taken as $\frac{pl^2}{24}$ and negative moments over the counterforts as $\frac{pl^2}{12}$. Expansion joints should be provided at 60 to 80 ft. centres. Spandrel walls should be provided with sufficient joints to avoid restraint of freedom of movement of arch. Details of suitable reinforcement are also given. Criterion of allowable foundation pressure is maximum settlement allowable, and difference of settlement between one abutment and another. For any appreciable settlement a three-hinged arch or a free span should be used. For weak ground, caissons, i. e., steel or concrete vertical cylinders with cutting edges may be used. Skin friction may be allowed for, when a good foundation does not exist at reasonable depth. Timber piles are only to be recommended when under water for their entire length. The Engineering News Formula, is quoted. Details of typical reinforced concrete piles are given, and the Dutch Formula for driving is quoted. General descriptions of the best pile group arrangements in various types of bridges are described.—JOHN E. ADAMS

WATER WORKS

Reservoir at Barming, Maidstone. *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 543-549.—The reservoir is an extension of one already existing and is being constructed as close as possible to the existing structure. It is designed to withstand the stress due to either structure being

full of water. Details are given of the design and method of construction of retaining wall, floor and roof.—JOSEPH MARIN

ARCHITECTURAL DESIGN

Colored architectural concrete in large country residence. JOHN K. BRANNER. *Concrete*, Oct., 1930, V. 37, No. 4, p. 23.—Dutch Colonial type of architecture and use of colored concrete in exterior of building and on grounds are features of Woodside, California, residence built for cement company head. House is on a hill, and between two wings there is a court facing a formal walled garden containing orange trees. Reinforced concrete gable walls rise above the roof, and are ornamented with scrolls of Dutch character. Floors are of structural concrete, laid on light fabricated steel joists and surfaced with oak and teakwood nailed to furring strips in the concrete. Roof is covered with dark grayish brown shingle tile. Colored concrete is used in floor of the enclosed court, laid out in checkerboard design in black and white. Balcony above court is supported by white concrete columns.—C. BACHMANN

Concrete plastics in Afghanistan. DAHL. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 880-2.—Preparation of ornamental plastics for new City of Dar-ul-aman in Afghanistan was very difficult because of lack of suitable stone materials. Quarries of European type are practically unknown in this country. Stones are seldom longer than 24 in. or wider than 6 in. and cannot be used for greater sculptures. Faced by these conditions, architect and artist Karl Maas undertook difficult task of producing desired plastics of concrete. Clay and gypsum for molds were difficult to obtain and the latter had to be burned in small wood-fired furnace. Cement was furnished by new cement plant in Kabul. Aggregates were carefully washed and prepared. Among the many plastics may be mentioned a fountain, ornaments and pillars of royal castle in Kabul. Plaster technique is well known in Afghanistan where kind of clay-stucco is used. Very old samples of their primitive art are still preserved.—A. E. BEITLICH

FIELD CONSTRUCTION

BRIDGES

Mid-Hudson bridge paved in alternate slabs to equalize cable loads. *Eng. News Record*, Oct. 9, 1930, V. 105, No. 15, p. 581-582.—Dead load stresses on the suspension cables supporting the Mid-Hudson bridge at Poughkeepsie, N. Y., were equalized during laying of concrete by carefully planned sequence of field operations. Concrete roadway across the 3,000-ft. structure is 30 ft. wide between curbs and 8 in. thick, heavily reinforced with built-up mats of ½-in. steel rods. This pavement, together with two 4½-ft. concrete sidewalks 3 in. thick, imposes a dead load of 4,000 lb. per linear ft. on the bridge. Concrete was laid in three strips, the center one being 9 ft. 4 in. wide and the two side strips each 11 ft. 1 in. wide. An industrial railway was used to deliver mixed concrete and to handle forms and steel. The center strip was first completed by working from center toward both ends of bridge simultaneously. The narrow-gage track was then shifted to the finished slab and paving again started from middle. In this case, however, alternate panels of the two outside roadways were poured, first on one side of center strip and then on other, leaving staggered gaps. These openings were then filled alternately so that during the whole paving operation weight of roadway was distributed uniformly between cables.—D. E. LARSON

The "Albert Louppe" bridge of reinforced concrete. A. COYNE, E. FREYSSINET. *Genie Civil* (France), Oct. 4, 1930, V. 97, No. 14, p. 317-334.—The bridge, recently completed, consists of three reinforced concrete arches of 566.13 ft. net opening each, crossing the Elorn some four miles out of Brest. The very long spans are the feature of this structure and were adopted, after much debating, to reduce number of piers and consequently the delays and hazards inherent in maritime construction. After laboratory tests of locally available quartzite deposits, this material was approved for concrete aggregate. Experience on the job, where erratic results were frequently obtained, proved it advisable to add a certain proportion of dune sand, which was fixed at 50 per cent on the basis of tests. This sand acts as a lubricant and corrects the grading, increasing the proportion of fines. This method proved extremely practical and did not increase the costs. Field tests were made on 8-in. cubes. No especial care was given to the making of the specimens, so that the results may be considered as minimum values. Concrete in the arches was subjected to mechanical vibration in large masses, and is naturally far superior to that of the specimens tested. The following strength values were obtained on test cubes of the five grades of concrete used: Grade 1 (used in approaches), 2600 lb. per sq. in. at 7 d.; 3620 lb. per sq. in. at 28 d. and 5400 lb. per sq. in. at 1 yr. Grade 2 (used in approaches, apron, arches), 3500 lb. per sq. in. at 7 d., 4480 lb. per sq. in. at 28d. and 6300 lb. per sq. in. at 1 yr. Grade 3 (arches), 3860 lb. per sq. in. at 7 d., 5000 lb. per sq. in. at 28 d. and 6650 lb. per sq. in. at 1 yr. Grade 4 (foundations in sea water using fused cement, i. e., ciment fondu), 3180 lb. per sq. in. at 1 d., 4310 lb. per sq. in. at 7 d. and 4940 lb. per sq. in. at 1 yr. Grade 5 (foundations in sea water, ciment fondu), 4030 lb. per sq. in. at 1 d., 4810 lb. per sq. in. at 7 d. and 5190 lb. per sq. in. at 1 yr. The proportions by volume varied in the five grades as follows: 0.9 to 1.04 cu. yd. gravel, 0.5 to 0.65 cu. yd. of sand and 660 to 880 lb. of cement. The "Abrams cone" or slump test was used, the slump being 5.9 in. for portland cement specimens and slightly greater for fused cement. This test, however, loses its importance when the concrete is worked by vibration, as resultant settling is independent of quantity of mixing water. The portland cement used in 1:3 mortar had a compressive strength at 7 d. of 3080 lb. per sq. in. and at 28 d. of 3860 lb. per sq. in. The 1:3 mortar of the fused cement had a compressive strength at 2 d. of 3900 lb. per sq. in.; at 7 d. of 4320 lb. per sq. in. and at 28 d. of 4550 lb. per sq. in. A total of 32,500 cu. yd. of concrete was placed on this job, requiring a total of over 1600 tons of steel reinforcement and about 2600 cu. yd. of forms.—M. A. CORBIN

New Wandau bridge over river Enns (Austria). JOSEF KREBITZ. *Zement* (Germany), Oct. 9, 1930, V. 19, No. 41, p. 964-7.—Old wooden bridge was replaced by single span concrete arch bridge crossing highway, river and railroad. Main span over river valley is 242 ft. long, center being 50.8 ft. above normal water level. Arch is 16.4 ft. wide at center and increases to 23 ft. wide at abutments; its thickness varies from 2.6 ft. at center to 4.6 ft. at abutments. Some steel reinforcement was necessary at center and abutments only, since arch is so designed that only compressive stresses occur. Two prismatic concrete columns, 23.6 ft. wide and 6.5 ft. thick, are at ends of arch. Openings for highway and railroad are 32.8 and 45.3 ft. wide and 30.5 ft. high. Abutments consist of mass concrete foundations on solid rock. Roadway has inclination of 1 per cent; it is 16.4 ft. wide at center of arch and 19.7 ft. and 20.7 ft. wide over side openings. Reinforced concrete flat slabs, 16.5 in. thick with top layer of bituminous material, were used for its construction. Concreting was carried out in 31 sections, divided by joints 11.8 in. wide. Aggregates of high quality were found in neighborhood of construction site. All invisible concrete surfaces were finished with a coating of Inertol to prevent penetration of water.—A. E. BEITLICH

Reinforced concrete arch bridge over the river Rench in Erlach, Baden (Germany). H. GLOCKNER. *Zement* (Germany), Aug., 1930, V. 19,

No. 33, p. 782-4.—Old highway bridge, which was insufficient for increasing traffic was replaced by concrete arch bridge with span of 67 ft. Abutments of old bridge were used for new structure. Roadway, which is of suspension type, is 17 ft. wide and consists of 7.8 in. thick continuous reinforced concrete slab which is supported by 13 horizontal beams. These are 11.5 in. wide and 19.7 in. high and are placed 4 ft. 9½ in. apart. Cross-section of arches is 32.3 in. by 21.6 in. Two horizontal beams which are 16 in. wide and 19.7 in. high connect centers of both arches. Bridge crosses river at angle of approximately 76 degrees, 61 ft. above high water level. High early strength portland cement was used for arch construction while road slab was made of ordinary portland cement. Special care was taken for exact proportioning of aggregates which were sieve analyzed at site. Mixture (1:5) consisted of 1.5 parts gravel of 1⅜ in. size, 1 part gravel of ⅝ in. size, 2.5 parts sand and 1 part cement. Strength properties of cement and concrete were carefully examined during entire construction period.—A. E. BEITLICH

BUILDINGS

Transformer station in Nuernberg (Germany). G. PFEIFFER. *Zement* (Germany), Sept. 4, 1930, V. 19, No. 36, p. 855-8.—Building is five-story skeleton structure of reinforced concrete made exclusively of high early strength cement. It contains four transformer units. Foundation consists of one continuous reversed Pilz-slab, which is from 19.7 to 23.6 in. thick and strongly reinforced. Floors of superstructure were constructed for loads of 573, 471 and 164 lb. per sq. ft. Provisions were made for two additional stories on top when enlargements are necessary. These are designed for 123 lb. per sq. ft. each. Interesting construction details are given.—A. E. BEITLICH

Modern park structures in Frankfurt a. M. (Germany). CRAEMER. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 920-2.—Building which serves as an open air restroom was constructed of reinforced concrete. Flat ceiling was made as Pilz-construction with thickened center portion. Projecting part of ceiling rests on four reinforced concrete columns. Concreting was done in planed forms; no finishing was necessary. A second structure of similar construction is described.—A. E. BEITLICH

New forming methods for small concrete buildings. F. SCHNEIDER-ARNOLDI. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 922-4.—A building material with almost ideal properties for small building construction is light concrete with pumice or tufeceous materials as aggregates. Only disadvantage is price of forms which is too high compared with relatively small volume of concrete. Improvements were made by using certain standard forms for building parts, which were built very often. A new type of forms is described. Wire nettings are mounted on iron frames and are used instead of wooden forms. In order to keep exact distance between two sides of these wire forms, connecting links are placed between them. Concrete is poured with plastic or earth-moist consistency. Great care must be taken to keep concrete wet during setting period, since open wire meshes allow faster evaporation of mixing water, especially in warm weather. Amount of concrete which falls through meshes is very small. Distribution of concrete in forms can be easily controlled. Openings of the meshes are about 0.6 to 0.8 in. and thickness of wire is between 0.039 and 0.060 in. No finishing of concrete surface is necessary. —A. E. BEITLICH

Concrete distributed by two towers connected by adjustable chute. *Eng. News Record*, Oct. 23, 1930, V. 105, No. 17, p. 661.—Use of a chute suspended from a high line between two towers at opposite ends of a long theatre building was the economic solution of a concrete distribution problem

encountered in Beverly Hills, Calif. A small corner lot which dictated the location of the concrete plant at the inside rear corner and a high tower section at front of building necessitated this unusual arrangement. The building is 230 x 130 ft. in plan and 78 ft. high except for front section, which is 120 ft. high. Near the concreting plant a hoisting tower 247 ft. high was erected. From this point it was necessary to distribute concrete to the front corners of the building 250 ft. distant. A 187-ft. secondary tower without hoisting equipment was erected near the front of the building. The two towers were connected by a 1-in. cable which supported a 12-in. metal chute on a 1:3 slope. Secondary chutes extending from the towers enabled concrete to be distributed to all parts of the structure.—D. E. LARSON

Corrosion of structural steel. (See PROPERTIES OF CONCRETE.)

DAMS

Mersey river hydro development. CLIVE W. CURRIE. *Canadian Eng.*, Sept. 30, 1930, V. 59, No. 14, p. 501.—Nova Scotia Power Commission has completed and put in operation three developments on the Mersey River, at Upper Lake Falls, Lower Lake Falls and Big Falls, with a combined output of 28,000 h. p. at minimum head. Upper Lake Falls development consists of an earth fill dam with concrete core wall, 49 ft. high and 1600 ft. long with concrete spillway 250 ft. long. At Lower Lake Falls one mile below an earth dam with concrete core wall 4000 ft. long was built. Concrete spillway section is 850 ft. long. Dam at Big Falls is also of earth with concrete core wall with concrete spillway section 600 ft. long.—G. M. WILLIAMS

Unique cutoff construction and arched foundation features of Rodriguez dam. *Eng. News Record.* Oct. 16, 1930, V. 105, No. 16, p. 600-604.—Building a cutoff wall from streambed downward as excavation was advanced through shafts and providing an arch to support four buttresses of the highest Ambursen-type dam ever built, were innovations in foundation construction used at Rodriguez dam, now under construction on the Tijuana river in Mexico. The arch span at intradosal springing line varies from 86 ft. at cutoff to 76 ft. at downstream end of structure. This decrease of span has advantage of providing wedge action which increases resistance to sliding and places heaviest buttress loads over shortest span. Arch rise varies from 28.2 ft. at cutoff to 32 ft. at other end of 225-ft. length of arch barrel. Arch barrel is designed to carry entire load of four buttresses to sound rock at sides. For symmetrical loading, crown of arch was placed halfway between two buttresses. Final arch computations including rib shortening and shear distortion, proved entire arch to be in compression and no bending reinforcement was used. Heavy reinforcing parallel to and directly under each buttress provides for any lateral bending in the arch. Maximum design stress for concrete was 700 lb. per sq. in.—D. E. LARSON

The Perak river hydro-electric power scheme. *Engineer* (England), Aug. 22, 1930, V. 150, No. 3893, p. 191.—Perak River Hydro-Electric Power Co., has built a hollow reinforced concrete dam of the Ambursen type on the Perak River in the Federated Malay States, forming a storage reservoir having an area of about 10 square miles with a catchment area of 2350 sq. miles. Dam was built slightly curved upstream to take advantage of good rock foundation. Spillway section furnishing a head of 63 ft. at low water, has a total length of 684 ft. and is divided into two sections by a sector gate with free opening of 100 ft. situated between piers near the center of the river. Spillway section consists of reinforced concrete slabs forming a continuous deck on upstream and downstream faces, supported by reinforced concrete buttresses 18 ft. on centers based with horizontal concrete struts in every second bay. Slabs are held in place by offsets and notches in buttress heads. Vertical contraction joints are

provided at each buttress and horizontal joints are arranged at intervals of 16 ft. between each section of the slabs. On the left bank the dam changes to the solid gravity type with same spillway curve as hollow section. This section was poured with vertical contraction joints extending down to rock foundation at 30 ft. intervals. River bed below dam is protected from scour by reinforced concrete aprons extending 100 ft. downstream.—G. M. WILLIAMS

MISCELLANEOUS

Wagon for distributing ready mixed concrete. *Engineer* (England), May 23, 1930, V. 149, No. 3880, p. 584.—The "Sentinel" Waggon Works, Ltd., is constructing a wagon for distributing ready mixed concrete. The mixer drum with a capacity of 5 cu. yd., is cylindrical in section with a conical discharge end, and is filled with internal blades. Drum is mounted on a double-geared, tipping model steam wagon, with tipping gear operated by an oil pump. The drum revolves at 10 to 20 r. p. m. either when the vehicle is in motion or at rest. Two tanks mounted above drum carry water, one for supplying water for the mix and the other for flushing the drum after discharge. In operation, weighed cement and aggregates are loaded into drum through the apex of the conical end, proper amount of mixing water is sprayed in, and mixer drum is rotated about 5 minutes before arrival at destination.—G. M. WILLIAMS

New experiences in American and German concrete construction practice. LUZ DAVID. *Zement* (Germany), Aug. 7, 1930, V. 19, No. 32, p. 750-5.—Author discusses opinions and experiences of American investigators on manufacture of concrete, which were presented at annual meeting of American Concrete Institute in 1930. Special discussion is given of methods for manufacture of ready mixed concrete. Advantages and disadvantages of central plant mixed concrete and central plant proportioned truck mixed concrete are characterized. Cost calculations are given for economic installation and maintenance of ready mixed concrete plants. Second part of article deals with experiences in winter jobs. Heating and protecting methods for raw materials and finished concrete are illustrated. Calculation and design of reinforcement must be made subject of more careful consideration.—A. E. BEITLICH

Cooling tower at Croydon. *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 555-57.—The most interesting feature from the point of view of construction is the arranging of the scaffolding for these hyperbolic cooling towers. The ultimate cost of the work largely depends on the scaffolding.—JOSEPH MARIN

Bonding new concrete to old. *Engineer* (England), Oct. 10, 1930, V. 150, No. 3900, p. 393.—The Department of Scientific and Industrial Research has conducted experiments to determine what precautions must be taken to insure a good bond between old and new concrete with the following results: (1) Laitance which forms a porous, chalky mass of non-coherent material must be removed. (2) If concrete is more than four hours but less than three days old the surface should be wire brushed and washed after removing laitance. (3) A ½-in. layer of mortar, without coarse aggregate, of the same composition as the new concrete should be applied before the new is placed. (4) If the concrete is more than 4 days old the surface should be chipped away, brushed and rinsed. This is followed by a slurry of neat cement, then a layer of mortar as in (3) before placing new concrete.—G. M. WILLIAMS

Full circle section of 17-foot tunnel lined in single pour. *Eng. News Record*, V. 105, No. 16, p. 610-612.—Cushman No. 2 tunnel, 13,000 ft. long and 17 ft. inside diameter, is located on the Olympic peninsula about 70 miles northwest of Tacoma, Washington, and will convey water from the Skokomish River through a spur of Olympic Mountains to develop a static head of 475 ft.

on hydraulic turbines of a 75,000-kw. plant at tidewater. A special tunnel form with pneumatic placing makes it possible to concrete full circle of lining 17 ft. in diameter and 40 ft. long in continuous operation. With this method of placing, copper water stops are required only in circumferential joints at 40-ft. intervals. The carriage for steel forms consists of two 5-ft. steel girders, 83 ft. long, supported on four end posts consisting of 8-in., H-section steel columns. Each girder carries along its upper flange a row of flanged wheels serving as rollers on which the circular steel form is advanced along the carriage when being moved ahead to new position. The cylindrical steel form is so constructed and hinged that it is readily collapsed for moving ahead and then expanded into position for placing concrete. All concrete is put into place by means of a 2-cu. yd. pneumatic concrete gun with a 6-in. delivery pipe about 80 ft. long. Thickness of concrete lining is specified as 15-in. average with 8-in. minimum.—D. E. LARSON

Selling concrete right from the mixer. *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 30-33.—The J. L. Shiely Co., producer of sand and gravel and crushed stone, St. Paul, Minn., has developed two plants for production of ready-mixed concrete. One plant adjoins sand and gravel plant and the other adjoins crushed stone plant, each one using material in which it is in close touch. A definite guarantee is made by the company to deliver a mix of specified strength; as for instance 1,500 lb. per sq. in. in 28 days for a 1:3:5 mix; 2,000 lb. for a 1:2:4 mix; and 2,500 lb. for a 1:2:3½ mix. The delivered prices for 1:2:4 concrete, which is mix used on most jobs (guaranteed to have a strength of 2,000 lb. at 28 days) is from $8.00 to $9.05 per cu. yd. on less than 100 yd. orders, depending on zone or distance from the plant. The 1:2:3½ concrete (2,500 lb. per sq. in. strength) is 15¢ per yd. more, and 1:3:5 concrete (1,500 lb.) is 75¢ per cu. yd. less. Orders of more than 100 cu. yd. carry prices 50¢ per yd. lower than those given on the price list. Heating in winter is charged for at rate of 40¢ per cu. yd. A discount of 5 per cent is allowed for cash or payment by tenth of the following month.—E. S. HANSON

Railroad box car with haydite concrete floor. *Rock Products*, June 7, 1930, V. 33, No. 7, p. 74.—Universal Atlas Cement Co., Buffington, Ind., has tried out a steel box car with a floor of haydite concrete. It has been found to have advantages over wooden floor for shipment of bulk cement, as power shovel or scraper works better on cement than on wood. The car was in a smashup which sheared the king pin of one truck but the floor was not injured. Floor concrete mix was 1 sack cement, 2 cu. ft. coarse haydite and 2 cu. ft. fine haydite. It was put on 4 in. thick but not anchored to steel floor, expansion joints and asphaltic fillers being placed between floor and sides. Round ⅜-in. reinforcing bars, spaced 6 in., running in both directions, were placed ¾ in. from bottom and wire netting was placed near top. Weight, about 36 lb. per sq. ft., or about two-thirds weight of ordinary concrete, and the cost was 35¢ per sq. ft. Advantages of the floor are greater strength than wood and better insulating qualities (when used in refrigerating cars). Disadvantages are greater weight and less nailability than wood. This is noticed with automobiles and like freight where cleats have to be nailed to bottom of cars.—EDMUND SHAW

Calibrating supports for exact centering of reinforcement. *Genie civil* (France), Sept. 27, 1930, V. 97, No. 13, p. 309-310.—The method recently invented by M. L. Ponsolle consists of small supports or clamps in the shape of a stirrup, star, disc, helix, etc., which are used on vertical and horizontal reinforcement alike and serve to keep the right distances between reinforcement and forms, as well as between individual parts of reinforcement. The method is successfully applied on a building under construction at Angers. Supports are made of very resistant metal which will withstand effect of working concrete, will not rust, and are furnished at moderate cost.—M. A. CORBIN

Concrete pipe testing machine. *Engineer* (England), Sept. 19, 1930, V. 150, No. 3897, p. 323.—A pipe testing machine has been made by Wm. Grice and Sons, Ltd., Birmingham. It consists of a platform built of steel sections mounted on four wheels. A hydraulic cylinder and ram, carrying a crosshead, is mounted on two steel columns. The ram and cross head are adjustable on the steel columns for any diameter pipe up to maximum specified. Load is applied through a hydraulic ram and pressure registered by means of a gauge.—G. M. WILLIAMS

Ferry on Lake Constanz between Constanz and Meersburg. THEODOR LUTZ. *Bautechnik* (Germany), May 9-23, and June 6, 1930, V. 8., No. 20-22-24, p. 259-9, 326-30, 350-3.—Difficulties were encountered during construction of harbors in Constanz and Meersburg. Quay is built of reinforced concrete. Three rows of concrete poles 12.6 in. square and 24.6 ft. long were driven. Poles of outer row are connected by dovetailed reinforced concrete piling. Piles are 23.2 in. wide, 6.3 in. thick and 20.7 ft. long. Distance between poles is 22.3 ft., and 4.9 ft. where expansion joints were placed. On this pile foundation rests reinforced concrete slab, 7.2 ft. wide, 2.5 ft. thick and about 328 ft. long with concrete railing on one side. Concrete piles were made at site of construction. Detailed description is given of arrangement of reinforcements, concrete mixtures, concreting and finishing of concrete.—A. E. BEITLICH

Attaining low cost in mixing and placing. CHAS. C. MOODY. *Concrete*, Nov., 1930, V. 37, No. 5, p. 23-24.—Office building for Inland Co., at Indiana Harbor, Ind., a 2000 cu. yd. job, with very low cost limits, was laid out and equipment provided. Concrete was divided into two parts: 500 cu. yds. in foundations and foundation walls, and 1,500 in the floor slabs. Sand and stone had two prices, one based on team delivery; the other on delivery by cars alongside of the building. Latter price was 35¢ per cu. yd. less. Delivery track was about 150 ft. from center of building. Crane with boom having drift of 40 ft., by which aggregate could be unloaded and dumped into hopper was available. Seventy feet remained between the hopper and building. Raising aggregate hopper from ground slightly higher than usual made it possible to place mixer closer to hopper and chute the mix directly into the hoist bucket, eliminating wheeling entirely. Hopper was put on extension legs sufficiently high to permit dropping the aggregates directly from measuring bins into mixer. Total labor cost was $104 each eight hours, and the average unit cost was close to 70¢ per cu. yd. Previously estimated cost of the concrete was about $1.50 per cu. yd., giving a total saving of about $1,600, from which were deducted cost of equipment handling. Total cost of concrete was thus only $1.19 per cu. yd.—C. BACHMANN

Finds pre-mixed concrete field made to order for gravel producers. *Pit Quarry*, Sept. 24, 1930, V. 20, No. 13, p. 58-60.—Keefner Concrete Co., Des Moines, Ia., has new ready-mix plant which utilizes aggregates procured, from the Des Moines River, by Hawkeye Co-operative Sand & Gravel Assn. which occupies an adjoining property. A drag scraper brings material from river bed into a sump whence it is pumped to the screen in the tipple. After washing and screening, the products are stock-piled. Beneath the stock-piles is a tunnel belt conveyor which delivers to an elevator which, in turn, delivers aggregates to respective sections of main hopper. Gravel batcher has a large dial scale with a dash-pot feature. Mixer is a 2-cu. yd. tilting unit. Concrete is delivered in bathtub bodies.—A. J. HOSKIN

Sioux City ready-mixed concrete plant was built for winter service. *Pit Quarry*, Sept. 24, 1930, V. 20, No. 13, p. 41-43.—Plant of L. G. Everist Co., Sioux City, Ia., is prepared to heat concrete in the winter season, loading it into delivery trucks at average temperature of 75° F. Live steam at a pressure of 50 to 60 lb. is fed into aggregates hopper above mixer through pipe with many holes. Water for concrete also is heated by a steam coil to a temperature of 180° F. When it does not pay to fire a steam boiler, oil-jet burner directs its

flame into mixer through a hole in charging chute. Cement bags are hand-trucked from cars, lifted by belt conveyor to batching platform, and emptied into batching hopper as needed. Bins for sand, gravel, and crushed stone discharge by gravity into batching hopper through lever-operated gates. Weighing is done with a 2-beam scale. Water is measured by a 100-gal. volumetric syphon tank. Concrete is dumped from 2-cu. yd. mixer directly into delivery trucks having bathtub bodies.—A. J. HOSKIN

Concrete units replace damaged brick-work in bridge pylons. *Concrete*, Oct., 1930, V. 37, No. 4, p. 17.—Walls of pylons on railroad viaduct, built 8 years ago with common clay brick in shape of hollow square and faced with Benedict stone, showed outward bulge due to expansion of brick backing, and replacement was necessary. First step was careful removal of cast stone top and facing. Material was lifted off with derrick, set on sidewalk, and carefully cleaned. In the meantime, brickwork was torn down and base of pylon cleaned. The 8 by 8 by 16-in. hollow concrete back-up units were set in 1:3 portland cement mortar, tempered with hydrated lime. Laboratory report on Van Houten units showed average absorption of 8.43 per cent; compressive strength of 1,280 lb. per sq. in. over the gross area and 2,040 lb. per sq. in. on the net area. Field test of clay unit showed absorption of 18.14 per cent.—C. BACHMANN

Nashville ready-mixed concrete plant stores combined aggregates. *Pit Quarry*, Sept. 10, 1930, V. 20, No. 12, p. 43-44.—Nashville Ready-Mixed Concrete and Supply Co., Nashville, Tenn., has plant with daily capacity of 400 cu. yd. New improvements will enable storage of six grades of aggregates in bins above mixers. Aggregates are handled by cranes to a hopper which feeds two bucket elevators, discharging into any of six compartments in a 250-ton steel bin. Bulk cement is unloaded to a ground hopper feeding a bucket elevator to a 2-compartment steel storage bin. A screw conveyor feeds from this bin to two 1-cu. yd. mixers. For one of the mixers, gravel is weighed in a 1-ton-capacity batcher, sand is measured in a 28-ft. inundator, and cement is weighed in a 1,000-lb. batcher. Materials drop into mixer hopper ready for next batch. Water is supplied from a tank alongside inundator. For other mixer, sand and gravel are handled by a 2-ton-capacity weighing batcher, cement by a 1,000-lb. weighing batcher, and water by a measuring tank. This second mixer prepares also lime mortar and cement mortar. Concrete and mortar are delivered in eight 3-ton trucks with agitator bodies.—A. J. HOSKIN

Delivering 60,000 cubic yards of centrally mixed concrete in month. R. P. BROWN. *Concrete*, Oct., 1930, V. 37, No. 4, p. 19.—Warner Co. of Philadelphia has established Central-Mix Concrete (CMC) as high standard by careful attention to all factors, including dependable delivery. Agitator type was decided best method of transporting and 15 four cu. yd. Clinton trucks were purchased at start. Fleet now comprises 85. Feature of equipment is reduction of entrained air. Unit consists of cylindrical tank horizontally mounted on truck chassis by trunnion bearings at both ends. Cylinder is kept constantly revolving so long as it carries concrete, by means of separate motor, there being no blades or paddles. "Certified concrete" is also furnished. Certification is made by independent testing and inspection company having absolute control of mix, checking of proportions and setting water tank. Inspector is stationed at each mixer, and verifies every batch. Inspection force checks sand and gravel at each mixer four times daily. At start of every job where "certified concrete" is furnished, inspection staff makes a set of not less than 6 standard cylinders and breaks them at 7 and 28 days. At Berks Street plant, sand and gravel are unloaded from barges to stockpile over tunnel feeding a 36-in. rubber belt conveyor which takes material to storage bins over mixers, which in turn feed measuring hoppers. Cement is purchased in bulk. One and one-half minutes mixing time is specified. Two Ransome 2-yd. mixers and one 3-yd. Smith tilting mixer are in constant operation being cap-

able of turning out from 1,500 to 1,800 cu. yds. daily. Orders are received over direct wire from main office after having been checked by engineering department. They are transmitted to concrete plant over a direct wire and to mixer control room over ticker tape, order stating mix, amount of concrete required, location of job, when delivery is to begin, and time between loads. As driver leaves mixer, his truck is sealed and drum started rotating. Product is sold either on strength basis or in accord with individual specifications. Deliveries are made on zone basis.—C. BACHMANN

Operates four pre-mixed concrete plants serving builders in St. Louis. W. E. TRAUFFER. *Pit Quarry*, Aug. 13, 1930, V. 20, No. 10, p. 39-45.—The Park Ave. plant of General Materials Co., St. Louis, Mo., is largest, with capacity of 1,500 cu. yd. daily. Its equipment includes locomotive cranes and 1-cu. yd. clamshell buckets, steel bins, screens, weighing batchers, with automatic dial scales, or weightographs, and three 2-cu. yd. mixers. Cement is added to the charge by hand from sacks. Fyler Ave. plant is the newest and all operations are mechanical with capacity of 900 cu. yd. daily. Sand and gravel are received in hopper-bottom cars and discharged into a long wooden bin with capacity of 350 carloads. Crushed stone is dumped by trucks into a concrete hopper whence it is conveyed into a section of main bin. Aggregates are delivered by conveyors to top of mixing plant and are fed, through a split chute, to two 4-compartment octagonal bins above mixers. Cement, unloaded in bulk from cars, by a power scraper, is elevated to bins with a capacity of 2,400 cu. ft. each. A screw conveyor feeds from these bins to a bucket elevator, to mixer department. Cement, sand and crushed stone are fed to two sets of weighing hoppers with dial scales. These hoppers discharge to batch hoppers of two 2-cu. yd. mixers. Clara Ave. plant resembles Fyler Ave. plant. Its capacity is 450 cu. yd. Aggregates are discharged from cars on a trestle to ground storage. Conveyors take the materials to a 4-compartment bin above the mixer. From its 4-compartment bin, cement is taken by screw conveyors to join aggregates which feed by gravity into weighing batchers which have dial scales and feed a 2-cu. yd. mixer. Branch St. plant is smallest with a daily capacity of 200 cu. yd. It has no mixers but prepares batches for mixing in transit. Aggregates are received in cars and unloaded by a crane to a 2-compartment bin with weighing hoppers. These feed to the truck mixers. Cement is received in bags which are opened into a small elevator discharging into a bin above the weighing equipment. A fifth plant is to be erected on Franklin Ave. at Twelfth St. The company operates 103 trucks.—A. J. HOSKIN

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Oct., 1930, V. 37, No. 4, p. 31.—Assuming a stairway of the type illustrated in the layout, the problem of form work may be considered as comprising construction of two reinforced concrete stair landings at levels intermediate between floors, with three short runs of stairs connecting the landings to each other and previously built floors. Stair landings are likely to be either a slab or of light-concrete beam—and slab construction supported on reinforced concrete posts; or several edges of the landing slabs may be supported on recesses in stair enclosure wall. Faces of top and bottom risers (at the connection with the previously built floors) are several inches in from the face of the hatchway opening in the floor construction. Form work for stair landings must be built first. The soffit panel is sloping panel that forms underside of the stair run. It is the first to be placed after form work for stair landings is in position. This panel, as a unit, consists of ⅞ in. lagging placed parallel to the risers, and is held by 1¼ by 4-in. cleats. Longitudinal joists are supported on 2 by 6-in. ledgers spiked to 4 by 4-in. posts. Following assembly of the soffit panel and its supports, form work consisting of stringers, beam sides and risers must be placed. On the free edge of the stair run there may be a small beam extending from landing to landing or from landing to floor. This will require a form for the sides and bottom of this beam. In the more usual case there is no

beam extending below the soffit, in which event the soffit panel extends to and beyond the edge of the stairs and only a beam side is provided. On the wall side the stair concrete is to be in contact with the wall, so that there is no room for a stringer or a beam side plank at the side of the stairway. A 2-by-8 stringer is placed against the wall, just above the risers, scabs again being nailed against the stringer to hold risers. In designing a mix for a stair run, a good plan is to use a 1:3 cement-sand mortar, or a concrete mixture with small coarse aggregates (not exceeding the ½ in. size) for that part of the slab below the steel reinforcement. The rest of the slab and the steps themselves may then be made of ordinary concrete of dry consistency. Another practice is to use small-sized coarse aggregate, either pea gravel or crushed aggregate not exceeding the ½ in. size, for all concrete in stairway work. Most convenient place to set posts and props supporting stairway form work is on newly concreted stairway and landings immediately below.—C. BACHMANN

Concrete plastics in Afghanistan. (See ARCHITECTURAL DESIGN.)

Construction of quay at Rotterdam. (See ENGINEERING DESIGN—MISCELLANEOUS.)

Exposed surfaces of concrete and their protection. (See PROPERTIES OF CONCRETE.)

RAILWAYS

Tokyo's underground railway. H. SAKAKIBARA. *Eng. News Record*, Oct. 23, 1930, V. 105, No. 17, p. 655.—An underground railway 8½ miles long is being constructed at Tokyo, Japan, for the Tokyo Underground Railway Co. The subway consists of a double or twin tube having rectangular steel frames of I-beams spaced 5 ft. apart and embedded in heavy walls, floor and roof of reinforced concrete. Concrete in the walls and roof was poured through chutes from a portable mixing plant on the street surface.—D. E. LARSON

ROADS AND PAVEMENTS

Concrete roads and their joints. H. C. JOHNSON. *Engineer* (England), Sept. 26, 1930, V. 150, No. 3898, p. 349.—A concrete road built in Cork, Ireland, 10 years ago and subjected to medium heavy traffic shows no defects. Section is 21 ft. wide, 7 in. thick at center and 5 in. at edges, flat base, with the edges reinforced with a concrete curb poured later but doweled to the slab, and reinforced with 2 ½-in. round bars. Transverse steel reinforcement in slab consisting of ½-in. round bars, 15-in. centers, with 7½-in. longitudinal distributing bars, is placed 2½ in. above lower surface of slab at center and rises to 2 in. from upper surface at sides. Concrete was a 20 per cent mixture, that is, 20 per cent of volume of finished concrete was cement. Slab was laid in sections 80 ft. to 90 ft. long, one half day's run with transverse shoulder or supporting lip formed by digging a shallow trench at the end of the slab. Vertical joint between slabs consists of tar paper attached to a ¼-in. thick pine strip. Concrete slab was roughly leveled with shovels, smoothed with long screed with double handles, operated transversely, followed by a light roller longitudinally, then finally by a 10-in. belt transversely.—G. M. WILLIAMS

Expansion joints in concrete. *Canadian Eng.*, Oct. 14, 1930, V. 59, No. 16, p. 544.—The Technical Advisory Committee of the Ministry of Transport of Great Britain has made a survey of defects in a recently constructed road three miles long, 30 ft. wide between curbs, built on a ballast bank varying in depth from 2 ft. to 20 ft. Wearing surface consists of 2-in. stone filled asphalt carpet on 8-in. thick reinforced concrete foundation. Concrete was laid in bays 14 ft. 6 in. wide, full width of road with no center joint, on the alternate bay system, every fourth joint filled with bituminous sheeting, with joint

supported on a 12-in. by 8-in. beam formed below. Mix was 1:2:4 by volume, ballast aggregate, with 4.7 lb. reinforcement per sq. yd. placed 2 in. from under side of concrete slab. Traffic consists of 10,000 tons per day mainly fast moving heavy commercial vehicles. After twelve months of service cracks have appeared at 14 ft. 6 in. intervals, indicating movement between adjacent slabs. Concrete has failed at the joints apparently due to depression of slab carrying load below level of adjacent slabs which in turn are struck heavy hammer blows as the load comes on. Remedy for these joint movements on newly made ground has not been definitely established but defects may be eliminated by thickening slabs at joints, or as a whole, or by the use of beams or interlocked joints.—G. M. WILLIAMS

Granili Street in Naples. LUIGI BRUSCHETTI. *L'Industria Italiana Del Cemento* (Italy), Feb., 1930, p. 20-24.—Granili St. is a concrete paved street built in 1925 which carries a very heavy traffic of iron wheel horse drawn wagons. Although some repairs have been necessary recently, concrete has generally shown little signs of wear. The author contends that the fault was in the materials or method of placing at particular points where disintegration has occurred. It is suggested that the surface of a concrete street should be made up of about 2-in. stone and cement without sand or intermediate size stone. A photograph is given showing a bridge roadway of concrete which has been in place for 22 years without deterioration.—C. G. CLAIR AND M. N. CLAIR

Results obtained by use of cement in Germany. VILBIG, MAIER, MULLER, OTZEN AND RIEPERT. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, Washington, D. C., 1930.—First concrete roads in Germany were built in 1890. Although methods of building these "cement macadam" roads differed appreciably from those in use today, many of these pavements are still in good condition. Extensive use of concrete as a paving material is limited by economic conditions. In general, this type is used only on heavily loaded roads, roads carrying a preponderance of motor vehicles and roads constructed in new locations. Two course method is preferred, using a 1:7 mix by volume in base and a 1:4 mix by volume in top. Carefully selected, durable materials are used in top and local materials in base. For bottom course, concrete has a consistency of moist earth whereas concrete in top course is somewhat wetter to facilitate finishing. Both hand and machine methods are employed but machine finishing is on the increase. Pneumatic tampers are used to consolidate base course. Steel reinforcing has recently been introduced using a 2½-in. mesh, placed about 1½ in. below surface. Slab thickness varies from 6¼ to 12 in., of which about three fourths is in the bottom course. Thickened edge sections are employed, with longitudinal joints on all roads more than 20 ft. wide. Transverse joints are spaced from 26 to 49 ft. apart, depending upon conditions and are the plain butt or open joint type, subsequently filled with asphalt. Sodium silicate has been used for hardening surface, without any marked improvement being noted. Bituminous treatments for curing have not been successful, although original laboratory results were favorable. Curing at present is by means of canvas followed by earth or straw as in the U. S. Curing period is in general from three to four weeks. Rate of progress attained in construction is variable. From 395 to 430 ft. of pavement 16½ ft. wide in 8 hours is considered good performance. Results secured using special Soliditit cement (cement to which ground silica has been added), have been good. Results may have been due to care exercised during construction. Limited results from use of Rhoubenite concrete have not been satisfactory. This concrete contains "Houben powder" an admixture. At first, concrete construction met with considerable opposition in Germany, as it was felt that there was a danger of destruction of the slab by formation of cracks and by steel tires of horse-drawn vehicles. These objections are no

longer of importance because of increase in automotive traffic. Considering the results obtained with concrete roads, the outlook for this type of construction is steadily and surely becoming better.—F. H. JACKSON

Results obtained by use of cement in Belgium. E. VAN HAUWERMEIFEN. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Cement has been used in Belgium in two forms, in concrete and mixed with stone dust to form a macadam binder. Most of the pavements have been built under the "Rhoubenite" or "Soliditit" patents, being constructed between 1921 and 1927. Maintenance was at first neglected but is being attended to much more promptly than in the past. Maintenance consists in filling cracks and joints with tar or bitumen. Light bituminous skin coats have not proved successful. Due to practice in Belgium of constructing the even numbered slabs after odd numbered slabs have thoroughly set (sometimes a period of several days elapses) it is difficult to obtain smooth riding surfaces. Author believes slabs 33 ft. long should be constructed rather than 16½ ft. which is present practice, so as to reduce number of impacts due to difference in elevation between adjacent slabs. In contrast with American practice, concrete pavements are constructed on a plane subgrade and are thickened at center. Proportions are controlled very accurately, coarse aggregate being measured in 3 sizes. Proportions now required are as follows: 1000 lb. portland cement, 12½ cu. ft. of sand, 9 cu. ft. stone chips, .08 to .2 in. in size; 10½ cu. ft. stone chips, .2 to .8 in. in size; and 21 cu. ft. crushed stone, .8 in. to 1.6 in. in size. A relatively dry mix is used and the concrete is consolidated either by rolling followed by hand tamping or by mechanical tampers. Construction is in two courses. When rolling is specified, slabs are constructed continuously, otherwise alternate construction is employed. Curing is done by covering with sandy soil kept moist for 25 days. Specifications require that the concrete develop a crushing strength of 5650 lb. per sq. in. at 56 days. Tests are made on 4-in. cubes sawed from 16 in. by 16 in. blocks which are chiseled out of the pavement. A penalty is exacted for failure to obtain this value which is calculated as follows: Penalty in francs per sq. cm. $\frac{400-X}{100}$ where X is the unit crushing strength obtained by test (expressed in kg. per sq. cm.). In order to compare the results obtained with practice in the U. S. 6 in. by 12 in. cylinders are cast alongside the road. Concrete cores are also drilled from the pavement to compare with cubes and cylinders. Approximately 124 miles of pavement has been constructed on the national roads. This type has won the unanimous approval of the road user due to its cleanliness, non-skid properties, visibility at night, etc. The one disadvantage is irregularity of surface due to difference of elevation of adjacent slabs. A satisfactory estimate of cost is difficult to obtain. It may be said in general that cost of construction is about one-half cost of block paving and about twice as much as ordinary surface treated macadam. The report includes a discussion of dry cement-stone dust mixtures as macadam binders as well as so-called mortar macadams. In latter type, a rich portland cement mortar is spread upon a course of well consolidated crushed stone upon which a second course of stone is placed and the whole thoroughly consolidated by rolling until the mortar flushes to the surface. Author concludes that mortar macadam, although low in first cost, does not form a durable pavement, it being usually necessary to surface treat within one or two years. Construction by this method has therefore been largely abandoned in Belgium.—F. H. JACKSON

Results obtained by use of cement in Denmark. S. ELLERT, E. SUENSON AND P. ORUM. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Report describes a number of experiments in use of cement in road construction since 1924, including construction of a concrete pavement in accordance with methods used in U. S. as well as several sections of so-

called mortar macadam. Results of the trial section of concrete have been excellent. The cost of this type is so high, however, as to prevent its adoption in place of Durax (stone block) with which it has to compete. Various trial sections with cement macadam have been built. The methods include the use of both a dry cement-sand mixture and a cement grout with which to fill the voids in broken stone course. Use of dry mortars was not satisfactory due to difficulty in filling the voids completely. The results with grout seem to be reasonably satisfactory and the cost is considerably less than ordinary concrete. Warning is sounded as to comparative load supporting capacity of cement-macadam road and concrete road. With increase in traffic by motor truck, the time is probably not far distant when intensity of such traffic will require construction of concrete pavements and it is intimated that immediate economy of using more temporary road surfaces will hardly be economical in the long run.—F. H. JACKSON

Results obtained by use of cement in Spain. JOSE RODRIGUEZ SPITERI AND FRANCISCO DE ALBACETE. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Results which have been secured on a number of short sections of concrete pavement constructed in various parts of Spain are reported. A typical pavement on highway from Madrid to France through La Junquera has the following characteristics: Width, 23 ft.; thickness, 9 in. Proportions: cement, 760 lb.; sand, 16½ cu. ft.; gravel, 29 cu. ft.; water, 4.2 to 5 gal. per 7 cu. ft. of concrete. Expansion joints were 48 to 64 ft. apart, and the pavement was cured by ponding, except a small stretch cured with Hunt process. This pavement was constructed in 1928 and is giving satisfactory service. More cracks are reported in section cured by the Hunt process than in the water cured section. Report also describes so-called mosaic concrete pavements as constructed in Spain. These roads consist of a concrete base, 6 in. thick, a cement mortar course 2½ in. thick and a top course of small stones with portland cement mortar binder. The diameter of the stones in the surface course varies from 3 to 5 in. on upper side and height is from 5 to 3½ in. The bottom area is 75 per cent of the surface area. These pavements are in reality stone block wearing surfaces embedded in a cement mortar cushion and laid on a concrete base. They appear to be giving excellent service under traffic carrying a substantial portion of steel tired horse drawn carts.—F. H. JACKSON

Results obtained by use of cement in France. R. BOUTTEVILLE. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Development of cement concrete road has been retarded in France by reason of high cost. At end of 1928, approximately 605,000 sq. yds. of the highway system had been surfaced, out of a total area of 1,030,000 sq. yds., including city streets. Indications are, however, that the amount of paving of this type will rapidly increase within the near future. As the result of test and experiment since the war, the methods of construction have become fairly well standardized. The report covers the various details of design and constructon as now practiced in considerable detail of which the following is a brief summary: Concrete pavements are either laid upon old stone road, reshaped or upon a foundation of stone or ordinary concrete. Thickness of slab varies, depending upon conditions, from 4 to 8 in., with a tendency to use the thicker sections. Pavements are in general of uniform depth, although an increased edge thickness of from 2 to 4 in. is sometimes used. Longitudinal joints are only used in pavements over 25 ft wide. Transverse joints are placed from 32 to 130 ft. apart with a tendency to the shorter lengths. Although recognizing the advantages of laying slabs in alternate sections from the standpoint of crack width, the practice has not been followed in France due to construction difficulties. The use of reinforcement is exceptional; only being used to increase resistance to bending on very unstable ground. Tests having shown that resistance to wear

can be greatly increased by increasing the amount of cement and also by careful selection of aggregate the old practice of one course construction has been abandoned and a two course type adopted with a proportion of approximately 550 lb. of cement to 17½ cu. ft. of sand and 35 cu. ft. of stone for the lower course as compared to a mix having 1200 lb. of cement (usually super-portland) to 10½ cu. ft. of sand passing a ¼-in. sieve and 35 cu. ft. of hard crushed porphyry, diorite, etc. Considerable attention is paid to the grading of the aggregates. Concrete is mixed in central mixing plants and transported to the road in small carts or wheelbarrows. About 40 gal. of water to 35 cu. ft. of concrete is used. The concrete is compacted by pneumatic or vibrating hammers or plates several types of which are described. The consolidated concrete is finished by means of a longitudinal roller weighing 1100 lb., about 18 ft. long and 10 in. in diameter. The methods of compaction described have resulted in much denser and stronger concrete with a perfect bond between the courses. The report estimates the life of concrete roads as now constructed to from 10 to 15 years. Attention is called to the fact that some of older roads are now in need of surfacing and describes a pneumatic device for scarifying worn concrete surfaces. The author does not believe that concrete roads should be built unless they can be constructed to resist surface wear. Need of more detailed specifications for concrete pavement work is discussed.—F. H. JACKSON

Results obtained by use of cement in England. J. B. L. MEEK, D. A. DONALD, S. H. MORGAN AND R. A. B. SMITH. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Report discusses the theory of designing concrete pavements as well as the materials used and construction methods followed in England at present time. Effects of temperature, moisture and traffic upon design are discussed and necessity of artificial drainage in order to lower moisture content of subsoil is stressed. The theory of loads is discussed to some extent, attention being called to the fact that loads as high as 30 tons per axle have to be provided for. Widening pavement 18 in. or 2 ft. beyond actual wheel path is practiced rather than thickened edge as used in the United States. Reinforcing is used on unstable soils, design varying with type and character of traffic. Slabs are usually reinforced both top and bottom. A minimum thickness of 8 in. is indicated for all main roads. Construction of slabs in longitudinal strips about 10 ft. wide is recommended with corresponding transverse joints 20 to 30 ft. apart in mass concrete and from 50 to 60 ft. apart in reinforced concrete. The alternate bay method is sometimes used, especially where it is desired to pave entire width at one operation and tampers and finishing boards cannot be used transversely. Due to necessity of providing a wearing surface of high resistance, pavements are usually constructed in two courses, using local materials in the base and crushed granite in the wearing course. The difficulty of applying a bituminous carpet which will adhere is discussed. The desirability of providing a carpet coat to take wear is noted however, and attention is called to the fact that application of a prime coat of tar or creosote will assist the carpet to adhere. Construction methods follow in general those used on the Continent. A dry mix is employed which is consolidated by tamping and cured by means of a 2-in. layer of sand kept wet. High early strength cements as well as, in certain cases alumina cements are employed when it is desired to open the pavement to traffic at an early age. The present day cost of concrete pavement in England varies from 8s to 10s per sq. yd., unreinforced but including excavating, etc. A method of constructing concrete roads known as the "sandwich" system is described. This consists in laying a course of broken stone, covering this with a plastic 1 to 2 mortar over which is spread another course of broken stone and the whole rolled with an 8-ton roller until the mortar flushes to the surface. This method has progressed well beyond the experimental stage and may be considered satisfactory. The cost varies from 5s to 7s per sq. yd. for a 4½ in. depth.—F. H. JACKSON

Results obtained by use of cement in Italy. L. DELL GUADIO, F. COLAMONICO AND I. VANDONE. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Use of concrete as a paving material in Italy dates back about 18 years. Concrete roads of the Soliditit type now comprise about 218,000 sq. yds. A notable development has been the use of concrete on the roads which are built to carry automobile traffic exclusively. A total of 1,570,000 sq. yds. have so far been constructed with about 1,210,000 sq. yds. to be laid in the near future. Concrete in "motor roads" is proportioned, mixed and placed in accordance with American practice. No steel tired traffic is, however allowed on these roads. For the main roads it has been found necessary to adhere to the European practice of two course construction, using a dry mix which can be tamped into place and hard wear resisting aggregates. A major portion of the report is devoted to a description of an experimental road constructed in 1929 which was built for purpose, primarily of developing means for constructing one half of the total width at a time so as to leave other half open for traffic. Other variables studied included the construction of alternate sections; proportioning by weight instead of by volume; using different grades of coarse aggregate; using 5 different cements; using various proportions of cement from 660 to 990 lb. per cu. yd., using various consistencies, using various consolidating and finishing methods, using single course as compared to two course work and using various thicknesses of slab. The above form 108 different combinations, each section being 15 ft. long and 11½ ft. wide. In order to accelerate the results of this test, an artificial steel tired traffic of 180,000 tons was carried over it. This did not produce enough wear to permit of a satisfactory comparison, which is now being obtained by means of very accurate level readings on plugs set in the pavement. These tests are being supplemented by profilometer tests to determine surface irregularities, in much the same way as is done in the United States. The results of this experiment are being awaited with great interest by the authorities.—F. H. JACKSON

Results obtained by use of cement in Holland. P. W. SCHARROO AND J. W. CLERY. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—This report, after calling attention to the inherent advantages of concrete as a paving material, calls attention to the fact that good results can only be secured when great care is used in construction. Only specialists and contractors who can provide proper personnel and good materials should be employed. Open bidding for this reason is out of the question. Concrete paving in Holland is still in beginning of its development. Unstable soils present great difficulties to this as well as other types of construction. During years 1925 to 1929 about 47 miles of concrete had been laid on the national system of roads as well as a small amount in the cities. The theoretical discussion of factors which affect density of concrete, importance of avoiding oversanded mixes is stressed as is also the necessity of using hard, strong and clean well graded aggregates. Use of dry mixes which have to be tamped is advocated in order to increase density. Ordinary portland cement will give as good results as super-cement. Although practically all roads in Holland are built in two courses, authors favor one course construction, on account of difficulty of properly bonding two courses together. Resistance to wear is of great importance and increases with strength and density. Water content should be kept low. Bituminous carpet coats have not proved satisfactory. Water glass is used as a surface treatment to increase hardness. Authors doubt the value of the treatment in case of concrete of proper density. Practically no reinforcement was used until 1928. Experiments, however, with longitudinal bars have indicated value of reinforcing of this type. Most recent practice as to joint construction has been to follow American practice, with tongue and groove longitudinal joint and transverse joints supported at ends with dowels fixed in one slab, but free to move in the other. Authors however,

are not entirely satisfied with this method, believing that joints should be so constructed as to leave slabs entirely free to move with respect to each other. They doubt that this is the case in American practice. Conclusion is drawn that concrete roads if properly built are better adapted to intense traffic of heavy vehicles than any other type.—F. H. JACKSON

Results obtained by use of cement in Sweden. E. PAUL WRETLIND. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Modern cement concrete roads now extend all over south portion of Sweden. During period 1925 to 1929 a total of approximately 15 miles was constructed, all single course construction. In 1926, a special specification covering a high grade portland cement was introduced. This cement known as Class A is used in public road construction. The requirements for strength at 7 days are almost double that of ordinary portland cement. Report describes an experiment with the alternate bay type of construction, in which openings 1.6 ft. wide were left at end of each 100-ft. slab and these openings filled with reinforced concrete about three weeks after the construction of the long slabs. In order to preserve alignment the reinforcing was doweled into the ends of the long slabs, with proper provision for slippage. Curing is done with wet sand, earth and straw. Sodium Silicate is used as a surface hardener without, however, any appreciable benefits.—F. H. JACKSON

Results obtained by use of cement in Switzerland. E. WYDLER. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Very few concrete roads were built in Switzerland before 1929, due, first, to high cost and, second, to fact that political sub-divisions of country into 25 Cantons is not favorable to establishment of a unified system of roads. Approximately 105,000 sq. yd. of concrete was laid in 7 Cantons in 1929. General practice is to lay concrete on old well compacted stone bases. First pavements were constructed in two courses, using a rich mix and very hard aggregate in the top course to minimize surface wear. Results are not satisfactory due to excessive cracking of the top course. Present practice is to use either one course or two courses, using same proportion in each. In accordance with continental practice, a very dry mixture is used which is either tamped in place with pneumatic tampers or finished with a finishing machine. It is concluded that longitudinal joints should not be used when width is less than 20 ft. Transverse joints should be constructed about 33 ft. apart. Trouble has been experienced with joints due to abrasion and impact. It is concluded that joints should be filled absolutely flush with the road surface with an unyielding material.—F. H. JACKSON

Results obtained by use of cement in the United States. FRANK T. SHEETS, ROY W. CRUM, E. M. FLEMING, A. N. JOHNSON, CLIFFORD OLDER AND R. E. TOMS. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—At end of 1928 there had been constructed in the United States on strictly rural highways approximately 59,000 miles of portland cement concrete pavement, or more than four times mileage which existed at end of 1920. During year 1928 alone, 8,756 miles were built. Such intensive construction has necessarily called for the rapid development of design and construction practices. This report outlines the present methods employed in design, construction, and maintenance of concrete pavements. Considerable advance has been made in structural design of pavements. Report calls attention to certain fundamentals which should be observed in construction of subgrade, and reference is made to new developments in design of cross section, as well as important features in proportioning of reinforcing steel and construction of expansion joints. The trend is towards intensive research as to effect of various characteristics of constituent materials on quality of concrete. Recent investigations indicate that characteristics of the aggregates have more influence on tensile and flexural strength of concrete than upon compressive strength.

The design of concrete mixtures by the water-cement ratio and mortar-void methods is rapidly gaining in favor. To secure benefits of proper design of mixtures, the ingredients must be accurately measured and water content closely controlled. These important features are described and references given to more complete information. Rapid strides have been made in development of equipment for handling materials and for mixing and placing concrete. Rigid inspection and modern equipment have aided in securing improved quality, smoother surfaces, and better production. The curing problem still needs much study. The rapidly increasing mileage of concrete pavement has stressed the problem of maintenance. Much study has been given organization, methods and equipment. Some of more important methods are described. One new development is the use of high early strength concrete for repair work.—F. H. JACKSON

Final conclusions reached by Permanent International Association of Road Congresses. *Proc. 6th Congress*, Washington, D. C., 1930.—(1) Cement is becoming generally used as a paving material and has many inherent advantages. Rapid hardening cements have special advantages in particular circumstances. (2) Cement has been used successfully in construction of cement concrete base courses for other surfaces, for cement concrete pavements, and for cement-bound macadam. (3) Cement concrete pavements and also cement concrete base courses protected by appropriate wearing surfaces are suited to heavy traffic. (4) Where a large volume of steel-tired traffic is encountered, if cement concrete is adopted as a pavement material, a two-course pavement with the upper layer composed of very hard aggregates should be used instead of a single-course pavement. (5) Single-course pavements have successfully carried maximum volumes of traffic and maximum wheel loads when the traffic was largely rubber tired. (6) Cement-bound macadam has been successful on roads carrying light traffic not inimical to the macadam type of construction. This method seems especially advantageous in locations where the conditions of drainage or exposure are unfavorable to the use of ordinary waterbound macadam. A protective wearing surface seems equally indispensable on cement-bound and water-bound macadam. (7) In designing cement concrete pavements and cement concrete base courses to be surfaced with other materials the resulting pavement should have equal load-carrying capacity or structural strength, when similar traffic conditions are to be met. (8) Competent engineering supervision of design, construction, and maintenance of cement concrete pavements is necessary to insure good results. (9) It is desirable that subgrades be uniform and stable. (10) Pavement slabs must be designed to carry expected loads. Edge thickening is advantageous as a means of producing an economic and balanced structural design for concrete pavement slabs. (11) Longitudinal and transverse joints are commonly used and must be designed to meet traffic, subgrade, climatic conditions, and shrinkage of concrete, but in view of the fact that a number of concrete roads have been successfully constructed without joints, it is advisable that further research should be made on the whole subject of joints and cracks. (12) Scientific design of concrete mixes and weight proportioning of aggregates represent the most modern practice. (13) Construction operations are performed mostly by machinery, with resulting lower cost and better workmanship. Value of a cement concrete roadway depends to a large extent on perfect execution of work and, in particular, on homogeneity of the concrete. (14) Thorough curing of concrete surfaces is essential. (15) Maintenance of concrete surfaces, properly constructed, is relatively simple and reasonable in cost. In particular, maintenance should comprise immediate filling with suitable material of expansion joints and of any cracks which may occur.—F. H. JACKSON

The experimental street of Binasco. ITALO VANDONE. *L'Industria Italiana Del Cemento* (Italy), Feb. 1930, p. 8-15.—An experimental section of

concrete road was constructed along Pavese St. in Binasco in connection with the study of the problem of developing a type of paving that could be placed over the existing substructure and which would economically resist the action of both automobiles and horse drawn traffic. About 180 sections 11.5 ft. wide and 14.8 ft. long covering 108 different conditions were poured. The variables included 5 different crushed stones, one gravel, 5 different cements, 4 proportions of cement, 2 to 6 different consistencies, 2 depths of paving and two types of paving (single and double layer). Concrete was placed in half road width sections, the other half of road remaining open to traffic. Concrete was mixed at a central point and conveyed to place by a narrow gauge railway. Very careful attention was given to control of various quantities entering into the mix. The sand and gravel were weighed. Gravel was separated into sizes to promote uniformity. Difficulty was encountered in controlling the water content with the usual water tank on the mixer. Samples of concrete were taken at time of pouring for laboratory test but it was considered that complex disintergrating action of the steel wheel animal drawn traffic could be only properly shown by a large scale test under real traffic. Numerous horse drawn wagons were therefore circulated over the experimental section for several days. The wear was concentrated in a narrow lane so that a condition equivalent to one year of normal traffic was simulated. Very little wear resulted except for the removal of the surface skin. Measurements of profile of pavement were taken by a special apparatus which makes a continuous record as the machine moves along. It is planned to repeat measurement at intervals to learn something about the way in which deterioration of pavement occurs. Economics of use of concrete roads in Italy is discussed and it is concluded that this type of road can be properly used when it costs no more than other types when all factors of cost including deterioration and maintenance are considered.—C. G. CLAIR AND M. N. CLAIR

WATER WORKS

Water tower in Hassloch i. Palatinate (Germany). *Zement* (Germany), Oct. 16, 1930, V. 19, No. 42, p. 999.—Foundation of structure consists of one circular reinforced concrete slab, 34.4 ft. in diameter, heavily reinforced radially and circumferentially. Cylindrical stair case (4.9 ft. in diameter), in which pipe lines for feed and discharge are located, is erected in center of foundation slab. Tank consists of two concentric containers, each of which has capacity of 132,000 gallons. Outer diameter of tank is 52.5 ft., its height is 24.6 ft. and depth of water is 23 ft. It is carried on a circle of 12 reinforced concrete columns connected by two horizontal concrete rings each placed at one third of total height of columns. Tank is faced outside with hollow cast concrete bricks to protect structure against temperature effects. Ceiling is 4.7 in. concrete slab. Total height of structure is 144.4 ft.; 390 tons cement, 1265 tons sand and gravel and 126 tons steel were used for its construction.—A. E. BEITLICH

SHOP MANUFACTURE

Achieving light-weight building unit with ordinary aggregate. D. R. COLLINS. *Concrete*, Nov., 1930, V. 37, No. 5, p. 20-22.—The standard size 12-in. product of block manufacturer at Cedar Grove, Wis., weights 7 lbs. less than the 8 by 10 by 16 in. unit he manufactured a year ago. Corresponding saving in weight has been made in the 10-in. unit. Saving is due to substitution of rectangular for oval cores—taking material out of the corners and thinning the webs. The manufacturer is making units 7¾ in. high by 15¾ in. long. His average 8 in., 10 in. and 12 in. units weigh from 42 to 44 lb., from 51 to 53 lb., and from 60 to 62 lb. respectively. When a thin-webbed unit is being made it

requires careful supervision of the mix and the manufacturing operation, and should be handled only in a high-grade plant having a reputation for high-grade products.—C. BACHMANN

Proposed German standard specifications for light cast stones made from pumice. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 879-80.—Three different types are specified: stones made from pumice aggregates with (1) hydraulic or dolomitic lime, (2) cement and other hydraulic binders and (3) with only or mostly cement or special cements. Form of units must be 10 in. long, 4.7 to 6.3 in. wide and from 2.5 to 5.5 in. high. Compressive strength must be for units made with lime, 170 lb. per sq. in.; with cement, 284 lb. per sq. in. and with special binders 427 lb. per sq. in. Average of 10 tests must be determined. Weight per cu. ft. should not be higher than 52.95 lb. Heat conductivity number for air dried units should range from 0.13 to 0.16 at 68° F. Units must stand freezing and thawing 25 times. Additional data are furnished explaining origin and geologic nature of raw material, chemical composition, porosity, physical structure and possibilities for application in construction practice.—A. E. BEITLICH

What is a fair market price for light-weight units? H. H. POTTS, *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 16-17.—The material considered is a concrete unit for backing and partitions, and may be of any of light-weight aggregates now on the market. Their value is considered in relation to clay tile and complete data worked out shows what the cost of concrete units would be to compete with clay units at various market prices: for example if 5 by 8 by 12 in. clay tile are delivered to the building site at a price of $65.00 a 1,000, then 8 by 8 by 16 in. light-weight concrete units are worth $170.00 a 1,000 delivered. The author then works the problem still further to show that light-weight units of this size can be sold at a price of $170.00 per 1,000 or 17¢ each and carry a profit of 2¢ each.—E. S. HANSON

Two-in-one concrete pipe plant at Beloit, Wis. *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 12-15.—The plant of the Illinois-Wisconsin Concrete Pipe and Tile Co. at Beloit, Wis., occupies the site of a worked out sand and gravel plant and is separated into two departments, one for making pipe up to 24 in. in diameter of the bell and spigot type and the other for making pipe from 21 up to 60 in. of the bevel joint type. Plant also manufactures larger sizes by hand. Handling of products has been worked out very efficiently by means of two travelling cranes and several small tractors with trailers, also tractors with caterpillar tread for pulling and pushing larger sections.—E. S. HANSON

Random ashlar walls. W. D. M. ALLAN, *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 34-36.—This concluding article of a series of three on concrete random ashlar describes the manufacture of the various concrete masonry units required for typical random ashlar wall construction. Almost any plant equipped to manufacture standard concrete masonry units usually can, with only a few changes in existing equipment, manufacture units which can be used in building random ashlar walls. For typical wall construction, seven sizes of units, in which the dimensions of the exposed faces determine the sizes, will permit the flexibility ordinarily required in designing random ashlar patterns. Inasmuch as concrete random ashlar walls usually are built as 4-in. veneers or 8 in. masonry walls, this discussion deals with manufacture of units 4 and 8 in. thick. However, the operations are practically identical for producing units having other wall thicknesses.—E. S. HANSON